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American Concrete Institute Publications Policy

Approved by the Board of Direction; Its Application Outlined by the Publications Committee

POLICY

It is the policy of the American Concrete Institute to encourage participation by its members and others in the work of extending the knowledge of concrete and reinforced concrete as a basis for improved products and structures.

To this end the Board of Direction has assigned to the Publications Committee the responsibility of selecting for publication such papers, committee reports, discussions and other contributions or parts of such contributions, as in the judgment of the Committee, seem to offer most of value in attaining Institute objectives within space requirements, consistent with budget limitations.

INTRODUCTION

The object of this statement from the Publications Committee is to assist ACI members and others in preparing papers, committee reports and other contributions which will be acceptable for JOURNAL publication in furtherance of the Institute's policy as set forth by the Board of Direction (see above).

The essence of that policy in reference to any contribution is to be found in a proper balance between two opposing factors: (1) the *cost* and (2) the anticipated *benefits*—each as seen from the viewpoint of the ACI Membership, insofar as the Publications Committee is able to interpret that view.

It is convenient to think of any such contribution as the record of an experience or the cumulative record of many experiences in design, construction, manufacture, or in research related to such activities.

It is desirable to think of an *acceptable* contribution as one which records only so much of an experience as is likely to be useful to many others toward improvement of practice for better products and structures.

SUBSTANCE AND TREATMENT

In general, a paper, committee report, discussion or fragmentary comment within the Institute's field *may* be acceptable—

If in substance: (1) it presents suggestions original in subject matter or treatment, based on adequate factual record, for the improvement of design, construction or manufacturing practice;* (2) it confirms the findings of others, provided such confirmation is not superfluous; or if it tends to revise or even to upset established ideas and practice, provided the new record is significant and convincing; (3) it reviews, digests, correlates and arranges conveniently for use many accumulated experiences of well defined scope.

If in treatment: (4) its scope and purpose are clearly stated, its items of information are well organized for reader assimilation and its conclusions adequately but briefly supported; (5) it is objective, avoids personalities, and is untainted by a plea of special interest; in short, if it is in good taste from the viewpoint of technical and professional interest; (6) of discussion of other contributions, the writer confines himself to the subject matter pertinent to the scope of the contribution on which the discussion is based; (7) as to form and physical arrangement it adheres to Institute JOURNAL usages.

THREE MAJOR CONSIDERATIONS

If a contribution meets the foregoing conditions its acceptability for publication will be determined as far as possible by the Publications Committee in relation to three other considerations: (1) probable interest of and likely service to membership; (2) budget limitations; (3) the maintainance of a publications program of balanced subject matter.

SUGGESTIONS FOR MEETING ACI JOURNAL NEEDS

In extension of the foregoing statement of basic publications policy, (approved by the Board of Direction) the Publications Committee here implements the general invitation to contribute to ACI work, with more detailed suggestions.

General

(1)—The aim of the Publications Committee is to select for the JOURNAL and for occasional special publications a maximum of useful information with a minimum of outlay for drafting, engraving, paper, printing and other attendant items of publishing expense—all within ACI budget allowances.

*"Originality" is to be understood as having reference to *publication* as well as to *authorship*. The Institute will not usually consider publishing what has been published. Exception is sometimes made in publishing a paper whose content is covered in a bulletin of the originating agency in greater length and detail than would be appropriate for the ACI JOURNAL.

(2) JOURNAL needs change with the times; interest veers sharply with every challenge of the unknown; occasional shifts of emphasis are desirable. The Board of Direction has sensed the need for more papers which supply "information for use". Information for use may be the combined product of laboratory research, of job experiment and of practical field application. Whatever its evolution it involves skillful translation of *what we know* into terms of *what to do* because of that knowledge. Papers of that type offer unquestioned opportunity for large service.

(3) Interests of the reading audience are of primary importance. That author gives most distinguished service who writes what that audience will read with greatest interest, attention, understanding, and most important of all, with the greatest sense of the significance of what is read, in terms of what the reader is trying to do—a contribution in which facts fit readily into a pattern for their use.

(4) A large project (whether in design, construction, manufacture or research), in its very size and complication of problems, is likely to be unique and of limited general interest. Its individual problems may be common to many undertakings and therefore of maximum interest. Thus it is often best to select for the subject matter of a paper some one problem (or several problems likely to be closely related in the experience of the readers) rather than to try to cover all aspects of an experience.

(5) The useful substance of a paper or of a paragraph is its prime asset. The larger the audience to which the substance is likely to be useful, the higher is the appraisal of the contribution. When valuable substance is set forth in an orderly manner, with space economy, it usually has a quick, easy road to acceptance. On the other hand, valuable substance may reside in a contribution less acceptably set forth; but, with the author's continued help and his acceptance of suggestions, his offering may often be made highly valuable.

(6) An advance outline of what a prospective contributor proposes to write, is sometimes a means by which the Committee offers suggestions which assist the author in preparing his paper most acceptably.

How contributions should be submitted

(7) All contributions offered with a view to publication by the Institute should be sent in *triplicate copies* (one original and two carbons) addressed: Secretary of the Publications Committee, AMERICAN CONCRETE INSTITUTE, New Center Building, Detroit, Michigan.

(8) All text and captions for illustrations should be typewritten, double-space, on one side only of 8½ x 11-in. sheets of white paper, leaving a blank margin of at least one inch.

(9) All contributions should be submitted with a view to their becoming, if accepted, the property of the American Concrete Institute.

(10) Consideration cannot be given to contributions submitted also to one or more other prospective publishers. (If published elsewhere a valuable paper may be briefed in the Current Reviews pages.)

Preparation of text

(11) In the organization of the contribution, typing of the text and selection of illustrative material, the author should try to visualize JOURNAL rather than manuscript pages, considering ACI usages and the rigid type pages into whose compass all published material must be brought.* Particularly is this advance visualization important in connection with illustrative material (see below).

(12) Write plain, straightforward, conventional paragraphs following a preconceived organization of the information to be imparted, sparing detail likely to be of interest to but few readers and giving full information likely to be of interest to many. Avoid unwarranted capitalization. Underscore only to indicate the use of *italic type* and employ it sparingly. Observe that casual and isolated references to quantity or measure are spelled out; that precise dimensions and quantities employ numbers and abbreviations—not symbols.

(13) Thorough organization of the subject matter should follow a careful determination of the exact scope and purpose of the contribution. Introductory sentences should contain a *brief* but nevertheless explicit statement of that scope and purpose; then the reader is at once oriented, knows the author's objective without having first to grope through several pages. Note emphasis on "brief". Do *not* write a long statement of the need for the particular study which provides the basis of the paper.

(14) Every paper should be preceded by a synopsis, not to exceed 250 words—preferably not more than 150. The purpose of the synopsis is two-fold: (a) In the original published form it arrests the interest of the reader who is "thumbing through". (b) Subsequently it becomes a basic reference to the contribution and what it contains. The synopsis thus provides the author's best means of increasing his reader audience. It should cover even more briefly the introductory "scope and purpose" and state in briefest outline the end product of information recorded.

(15) The "story" should be told in the text—briefly but with such completeness in essentials that all tables, graphs and photographic illustrations are in the nature of parenthetical elaboration of detail best recorded in these mediums. Attention may be called to them in the text by a simple parenthetical reference, as: (Table 1) or (Fig. 1 and 2).

*See JOURNAL pages of September 1941, or later.

(16) Avoid as far as possible the use of words or expressions peculiar to a specialization. Avoid colloquialisms, job jargon and terminology whose use is unduly restricted—professionally, geographically or otherwise. Aim to be understood by young engineering students and by readers who *think* in other languages than English. Stay close to the common language of allied professions, arts, trades.

(17) Extended quotations from generally accessible literature are undesirable; use references. If few, they may be casual in the text, as "Talbot reports"—each such reference keyed by symbol (as *) to a specific footnote citation; if numerous, numbered (as [1]) to correspond to a list of references in a bibliography at the end of the paper.

(18) Tabulations should be numbered consecutively in the order of the first text reference to them. A table should not be inserted in the text following some such phrase as "in the tabulation which follows". Frequently it is mechanically impossible to place a table as so indicated. To split a table between two pages is undesirable and usually becomes necessary only when it *must be* more than a page long.

(19) In recording extensive test data in detail remember two things: (a) At this point the reading audience dwindles to the analytical few; (b) Composition for tabulations in 6-point type costs about six times as much per page as straight text. Test data may often be summarized in text or in compressed tabulations of "average values" and provide ample basic information for most readers. To provide the "complete record" which the research man likes to make available for purposes of check upon his technique and findings, extensive test data, likely to be of interest to but few readers, might often be made available to those few through the Institute in mimeograph or otherwise at cost of reproduction and so announced in a footnote. The Committee is well aware that in spite of the disparity in mechanical costs, useful tabulations often provide the most economical way of presenting much information in small space. It is merely suggested that tabulations be kept to essentials.

Illustrative material

(20) With no allowance for additional drafting, a line-cut from the usual line drawing costs more than two and one-half times as much as an equivalent text area. A half-tone illustration from a photograph, wash drawing or any illustrative material having essential variations of tone, costs about three and one-half times as much as the displaced text. Illustrative material should be selected to serve a purpose commensurate with its much greater cost—which well chosen illustrations justify. Photographs and drawings are all "figures" comprising one series numbered consecutively in the order of first text reference. Captions for

illustrations should not be lettered on photographs or drawings but should be supplied on a separate sheet of paper, listed consecutively in the order of reference for type composition.

(21) Most photographs and many drawings show too much—thus too little of essentials. To convey an idea, as of some construction feature, the author should aim to "play up" the important detail and eliminate any clutter of distracting non-essentials. Generally, a drawing should be specially made for the paper of which it forms a part, to show some interesting detail—nothing else.

(22) Photographs (glossy prints preferred), unmounted, and mailed flat should be protected against damage. Do not use clips.

(23) Graphs, diagrams, etc. should not be submitted on fine-line cross-section paper. Rarely will it stand necessary reduction without "smudge." The neat and painstaking author is prone to over-emphasize harmony throughout a series of drawings—a harmony related only to the contribution in its manuscript form. A "curve" which might be shown in very small space often occupies a grid sheet of the same size and with the same lettering as a more complicated graph subject to less reduction. Such dissimilar graphs are too often "framed" with neat, uniform borders—a comely job in manuscript but unsuitable for reproduction. Look at the printed page; visualize the minimum space in which a graph will convey essential information. Then make it with reference to necessary reduction (perhaps double or treble the probable final printed dimensions) and the lettering in dimensions chosen for legibility *after* reduction. If lettering is clean and open it is still legible at a height of about 1/16 in. It makes for convenience and economy when a series of graphs has a uniform reduction multiple. When the contributor is not sure of himself in the matter of his lettering it is better that it be put on lightly in pencil and left to an Institute draftsman. A graph should bear its proper explanatory legends; but explanations of and comment on other detail should be kept off the drawing, to be embodied in type captions, or if an essential of the "story", then in the text. Excessive lettering greatly complicates drafting and engraving problems, in unnecessary limitations on reduction. Folded inserts are not published.

(24) Not only should illustrative material be unmounted but especially should the author refrain from making pasted-up layouts which are not always in keeping with the final plan.

Discussion

(25) Note at end of each paper or report as published (and on the inside of each JOURNAL cover) the announcement of time limit for discussion. All published papers and reports of a technical nature in the

Institute JOURNAL are open to discussion. Discussion of discussion is the exclusive privilege of the original author of a paper discussed. Discussion is subject to all of the requirements of primary contributions and this additional one: Discussion must confine itself to the scope of the contribution upon which it is based.

Job Problems and Practice

(26) The JPP Department of the JOURNAL provides wide latitude in both subject matter and treatment—prescribes few formalities. It will be used more and more for brief contributions, for comment and criticism, and for attention to special problems and inquiries of some general interest. It is anticipated that the contributor will accord the Committee reasonable latitude in putting fragmentary contributions containing helpful substance into acceptable form for publication.

RELATIONS OF CONTRIBUTOR AND COMMITTEE

(27) In the Contributor's relations with the Institute's Publications Committee the contribution is a confidential matter up to the time of its final availability for publication—confidential within the limits of the procedure necessary to determine availability. That procedure follows this pattern:

(28) The Secretary-Treasurer of the Institute, in his capacity as Secretary of the Committee, advises the Committee, by memorandum, of a contribution received for consideration, giving title, name of the author, the synopsis, and an estimate of JOURNAL space requirement of text, tables, and illustrative material if published approximately as submitted. Such a memorandum implies that members of the Committee will give preliminary suggestions for consideration or disposition of the contribution, naming critics especially competent to consider it. Rarely is a paper passed for publication without critical examination by at least three persons. If there is unanimity of opinion as to the general acceptability of a contribution, action is usually taken based upon such a consensus. If there are wide differences of opinion other critics are called upon. Action is eventually taken by the Committee Chairman and Secretary (the Committee's executive group) in conformity with majority opinion—or in doubtful situations, the entire Committee may be asked to decide on the basis of critics' opinions.

(29) If it is a simple matter of acceptance or rejection of the contribution as submitted the procedure requires less time than is necessary when critics find the contribution acceptable subject to modification—parts eliminated as non-essential; the addition of further information; specified statements to be qualified. Rarely is a contribution accepted precisely as presented. Usually the contributor accepts suggestions (sent

to him in some detail) with thanks. If there are some residual differences of opinion, as between Author and Committee, concessions are likely on one side or the other, usually both. Further, the Committee reserves to itself certain purely editorial prerogatives (as in rearrangement for greater clarity) and even in these matters the author has final opportunity to approve or disapprove. If differences cannot be composed the Author is at liberty to withdraw his paper.

(30) During the process of manuscript consideration the Secretary of the Committee can at first only acknowledge its receipt, indicate that it has been referred to the Committee. If the contribution is acceptable, actual scheduling for publication depends upon factors of timeliness, prior commitments and possibly other considerations.

(31) For discussion of papers the procedure is usually (not always) a shortened form of that indicated for original contributions.

THE CONVENTION PROGRAM

(32) Most of the subject matter of the annual convention program (for which the Publications Committee is responsible) is selected from among papers and reports either already published or scheduled for pre-convention publication. They are selected for their adaptability to the special uses of a convention. A convention contribution is first chosen for publication on the basis of the criteria which have already been outlined. Its adaptability for a convention—an occasion when men come together "to discuss"—often requires special re-treatment. This recasting of subject matter is for that brief, simplified, oral presentation which a convention demands; the elimination of elaborate and confusing detail and assurance of opportunity for and participation in discussion—the participation of numerous individuals. A close line-for-line reading of long technical papers is often a deadly antidote for convention interest and spontaneity. (See "Technical Tedium or Otherwise" by R. W. Crum, April 1941 ACI JOURNAL. Separate prints on request.)

IN CONCLUSION

(33) It is well perhaps to repeat that the primary criterion for the acceptability of any contribution is its probable usefulness to the membership of the Institute. It is published to inform the reader, through such a record as is new, significant, noteworthy and unincumbered by what is already common knowledge or by that which is of such isolated application as to have little general interest.

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The Influence of Vibration, Consistency, and Grading of Aggregate Upon the Design of Concrete*

By G. W. HUTCHINSON†
Member American Concrete Institute

SYNOPSIS

Data in this paper supplement those previously published: "Concrete Aggregate Development on the Claytor Hydro Project." They are from the results of several investigations in the Claytor laboratory of factors affecting the design of concrete which could not be fully reconciled with existing theory. The more important indications:

1—That an optimum gradation exists for each aggregate or set of aggregates, when of given maximum size and combined with a given paste content, which provides maximum workability in concrete. Gradations of aggregate either finer or coarser than the optimum require additional mixing water in order to maintain equal workability.

2—To maintain a given consistency of concrete, a greater water content, per unit volume of concrete, is required as the paste content is increased.

3—The compressive strength and durability of concrete, especially in the mixtures sufficiently dry to require placement by vibration, are affected by the consistency of the concrete in a manner not definitely related to the water cement ratio except when the cement content is constant.

INTRODUCTION

In connection with concrete studies for the Claytor Dam, Pulaski County, Virginia,‡ numerous series of tests were made as a means to study the effects of different factors influencing the strength and other properties of the concrete. Some of these tests led to somewhat new conceptions of the parts played, even by such fundamental things as consistency and grading of aggregate. These roles became apparent in the endeavor to enhance, by any means possible, the quality of lean concrete of relatively dry consistencies of which large dams are built.

*Received by the Institute March 3, 1941.
†Concrete Technician for the Clayton Hydro Project.
‡G. W. Hutchinson: Concrete Aggregate Development on the Claytor Hydro Project. ACI JOURNAL, January 1940; *Proceedings* V. 36, p. 273.

They were perhaps emphasized by the rather unique properties of the artificial sand used—crushed dolomite reshaped by special milling to produce an unusual degree of workability, comparable to that of good natural sands. Some of these relations seem important enough to be taken into account in the design of concrete for major projects. In any case, they seem to disclose definite conditions under which there are occasional wide departures from the generally accepted water-cement ratio law.

If these deductions, together with the supporting evidence, as presented in this paper, are not fully accepted by those who care to study them, they at least suggest that well established notions regarding the internal structure of concrete will bear occasional re-examination from both the old and newer points of view.

OUTLINE OF TESTS

This discussion is based on five series of tests, the data of which are tabulated without unnecessary detail in Tables 1 to 5. The first four of these involve the same type and grading of coarse aggregate, but different types of fine aggregates.

Series X-1 (Table 1) constitutes a group of tests in which the cement ranged from 0.5 to 2.0 bbl. per cu. yd. A natural sand was used as fine aggregate. It was supplemented by additions of minus 100 mesh dolomite in such quantity that all mixes contained an amount of cement plus fine dolomite, equivalent to 2 bbl. of cement per cu. yd. on an absolute

TABLE 1—EFFECT OF CONSISTENCY AND WATER-CEMENT RATIO UPON THE COMPRESSIVE STRENGTH OF CONCRETE

SERIES X-1

Coarse Aggregate: Crushed dolomite, ⅜-¾ in.

Fine Aggregate: Natural sand B and crushed dolomite passing No. 100 sieve. Column 3 gives the amount of fine dolomite as the equivalent of bbl. of cement per cu. yd. on an absolute volume basis.

Test Specimens: 3 x 6 in. cylinders. Each value of strength is the average of five tests.

Mix	Cement bbl./ cu. yd.	Fine Dolomite	Slump (in.)	W/C by wt.)	Water gal./ cu. yd.	Compr. Str., psi.		
						7 Day	28 Day	90 Day
A	0.50	1.50	5½	1.72	39.1	469	651	83)
B	0.75	1.25	5¾	1.17	39.7	743	1290	1550
C	1.00	1.00	6	0.91	40.9	1320	1970	2560
D	1.25	0.75	6	0.73	41.4	1630	2330	3480
DD	1.25	0.75	4	0.68	38.7	1900	3110	4400
DDD	1.25	0.75	2	0.63	35.7	2490	3990	5410
DDDD	1.25	0.75	0	0.53	30.1	3410	4490	6730
E	1.50	0.50	6	0.62	42.0	1970	3200	4290
F	1.75	0.25	6	0.54	42.6	2430	3890	5160
G	2.00	0.00	6	0.48	43.3	3220	4220	6040

TABLE 2—RELATION BETWEEN COMPRESSIVE STRENGTH OF CONCRETE AND FINENESS MODULUS OF AGGREGATE

SERIES X-7

Cement Content: 1.20 bbl. of cement per cu. yd. of concrete.
Water Cement Ratio: 0.62 by weight.
Aggregates: Stone sand A and four natural sands, B, C, D, E. Coarse aggregate: ⅜ to ¾ in. crushed dolomite.
Consistency: Stiff, suitable for vibrating.
Test Specimens: 3 x 6-in. cylinders. Each value of compressive strength in the table is the average of 10 tests. Age 28 days.

Aggregate Ratio, Fine : Coarse	Fine Aggregate Identification				
	A	B	C	D	E
	Compressive Strength, psi.				
3 : 7	2600	2280	2260	2190	2550
4 : 6	3460	2700	2710	2530	3130
5 : 5	4730	2940	3150	2700	3400
6 : 4	5840	2750	3040	2810	3300
7.5 : 2.5	5270	2620	1240	1460	765
	Fineness Modulus of Total Aggregate				
3 : 7	5.76	5.76	5.76	5.60	5.54
4 : 6	5.35	5.35	5.35	5.13	5.05
5 : 5	4.94	4.94	4.94	4.67	4.55
6 : 4	4.52	4.52	4.52	4.21	4.07
7.5 : 2.5	3.92	3.92	3.92	3.51	3.34

TABLE 3—VARIATION IN COMPRESSIVE STRENGTH AND PLACEABILITY OF CONCRETE WITH CHANGE IN AGGREGATE GRADING

SERIES X-7B

Cement Content: 1.20 bbl. of cement per cu. yd. of concrete.
Aggregate: Stone sand A, 0-4; Crushed dolomite ⅜-¾ in.
Stone sand F, 0-4; Crushed dolomite ⅜-¾ in.
Test Specimens: 3 x 6 in. cylinders. Each value in the table is the average of 6 tests. Age 28 days.

Sand A	Aggregate Ratio Fine : Coarse	Fineness Modulus	Placeability (Seconds)	Compressive Strength psi.	W/C (by wt.)
1-A *	3 : 7	5.76	16	1760	0.62
2-A	4 : 6	5.35	12	3780	0.62
3-A	4.5 : 5.5	5.14	10	4470	0.62
4-A	5 : 5	4.94	9	5440	0.62
5-A	5.5 : 4.5	4.73	9	5680	0.62
6-A	6.5 : 3.5	4.32	15	5610	0.62
7-A	7.5 : 2.5	3.92	20	5590	0.62
Sand F					
1-F	3 : 7	5.76	15	2100	0.69
2-F	4 : 6	5.35	13	3200	0.69
3-F	4.5 : 5.5	5.14	11	3730	0.69
4-F	5 : 5	4.94	11	4040	0.69
5-F	5.5 : 4.5	4.73	12	3960	0.69
6-F	6 : 4	4.52	15	4000	0.69
7-F	6.5 : 3.5	4.32	20	3760	0.69

*All specimens honey-combed about one-half of total vertical area.

volume basis. The test results obtained from this series indicate: (1) that even in fairly wet mixes, suitable for hand placing, the water required to maintain a given consistency is not constant, but increases with the cement content; (2) that strength depends not alone on water-cement ratio, but on consistency as well. Durability of these concretes is discussed in the Appendix.

Series X-7 (Table 2) involves tests of concrete made with different sands at vibrating consistency. The cement content and water cement ratio remained constant. A group of tests was made for each sand with a progressive change in the ratio of fine to coarse aggregate. Comparison of compressive strengths with fineness modulus of aggregate show: (1) that the strengths vary widely with change in grading of the aggregate even when the W/C is constant; (2) that the maximum strengths occur at relatively low values of fineness modulus.

Series X-7B (Table 3) is similar to Series X-7, except that the tests are confined to two stone sands, one the specially milled sand and one coming direct from the primary stone crusher, the latter having many flat and elongated particles. These concretes made with each sand were of the same cement content and W/C, and of stiff but generally placeable consistency with the vibrator. Placeability* was measured by a special device for indicating the relative workability of mixtures having zero or near zero slump. These tests show markedly the increase in compressive strength with decreasing fineness modulus and they show that strength is near the maximum at maximum placeability.

Series X-12 (Table 4) shows compressive strength-fineness modulus relations for the reshaped dolomite aggregate at 3 different cement contents and water-cement ratios, all at vibrating consistency. In all groups, maximum placeability corresponds nearly to maximum strength, regardless of cement content, and in all cases the highest strengths are associated with what would ordinarily be regarded as over-sanded mixtures.

The fifth series of tests (Table 5) is a study of the placeability of different concretes of vibrating consistency, of constant cement content and W/C, as the ratio of fine to coarse aggregate varied progressively. The tests indicate that particle shape and surface characteristics of the fine aggregate have considerable effect on workability in concrete of this type.

MATERIALS

Sufficient normal cement to complete each investigation was secured in advance and properly stored. Coarse aggregate was crushed dolomite

*Description published in A.S.T.M. Bulletin, August, 1939.

TABLE 4—RELATION OF PLACEABILITY AND STRENGTH OF CONCRETE TO FINENESS MODULUS IN MIXTURES OF DIFFERENT CEMENT CONTENT

SERIES X-12

Cement content as indicated. Water-Cement Ratio constant for any given cement content.
Aggregate: Stone sand A, 0-4; Crushed dolomite, ⅜-¾ in.
Consistency: Stiff, suitable for vibrating.
Test Specimens: 3 x 6 in. cylinders. Age 28 days. Each value in the table is the average of 5 tests.

	Aggregate Ratio Fine : Coarse	Fineness Modulus	Placeability (Seconds)	Compressive Strength psi.	W/C (by wt.)
Cement Content: 0.95 bbl. per cu. yd.					
AS-1	3 : 7	5.76	45		0.83
2	4 : 6	5.35	12	2440	0.83
3	5 : 5	4.94	8	3190	0.83
4	6 : 4	4.52	7	3550	0.83
5	7 : 3	4.11	11	3630	0.83
Cement Content: 1.25 bbl. per cu. yd.					
BS-1*	3 : 7	5.76	15	2140	0.62
2	4 : 6	5.35	9	4270	0.62
3	5 : 5	4.94	6	4920	0.62
4	6 : 4	4.52	7	5220	0.62
5	7 : 3	4.11	10	5230	0.62
Cement Content: 1.75 bbl. per cu. yd.					
CS-1	3 : 7	5.76	15	5240	0.46
2	4 : 6	5.35	11	7040	0.46
3	5 : 5	4.94	8	7810	0.46
4	6 : 4	4.52	13	7850	0.46
5	7 : 3	4.11	20	8080	0.46

*All specimens honey-combed about one-half of total vertical area.

TABLE 5—EFFECT OF TYPE AND GRADING OF FINE AGGREGATE ON PLACEABILITY (WORKABILITY) OF CONCRETE

Cement Content: 1.00 bbl. pr cu. yd. of concrete.
Water-Cement Ratio: 0.70 by weight.
Coarse Aggregate: Crushed dolomite, ⅜-1½ in.

Aggregate Ratio, Fine : Coarse	Fine Aggregate Identification				
	A	B	C	D	E
	Seconds Required for Maximum Compaction				
3 : 7	25	18	26	18	15
3.5 : 6.5	15	8	16	9	11
4 : 6	11	5	10	8	10
4.5 : 5.5	9		14	13	13
5 : 5	9	5			
6 : 4	8	6			
7 : 3	16				
	Fineness Modulus of Total Aggregate				
3 : 7	6.11	6.11	6.11	5.95	5.89
3.5 : 6.5	5.88	5.88	5.88	5.70	5.62
4 : 6	5.65	5.65	5.65	5.44	5.35
4.5 : 5.5	5.42		5.42	5.18	5.08
5 : 5	5.19	5.19			
6 : 4	4.70	4.70			
7 : 3	4.27				

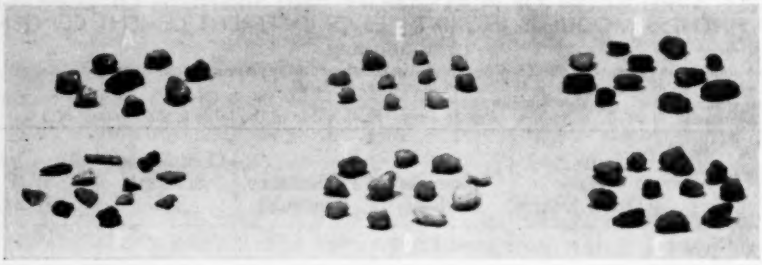


Fig. 1—Particle shape and surface characteristics of fine aggregate.

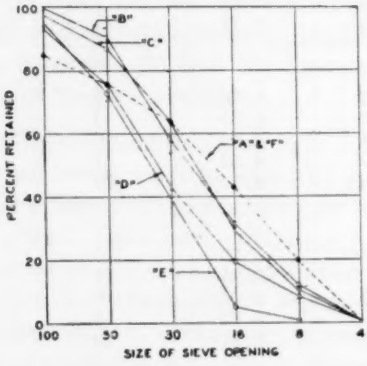


Fig. 2—Grading curves of fine aggregates

screened to pass a ¾-in. and to be retained on a ⅜ in.-opening. This was used for all series reported herein, except for the placeability studies summarized in Table 5, in which the size range was ⅜ to 1½ in.

The fine aggregates, with the exception of those used in series X-7 (Table 2) were separated into three sizes, 0-50, 50-16, 16-4, and then recombined in definite proportions for each investigation. The character of the different sands is shown in Fig. 1—typical particles of each, A and F being the dolomitic sands (prepared and unprepared, respectively), B and C relatively coarse natural sands, and D and E relatively fine natural sands. The gradation is shown in Fig. 2, in which attention may be called to the relatively limited amount of 16-50 material in the dolomitic sands, and to the relatively high fraction (about 17 per cent) passing the No. 100 sieve.

CASTING OF SPECIMENS

All concrete batches were mixed by hand, sufficient for making from four to seven 3 x 6-in. cylinders. Those of approximately 2-in. slump or less were cast by vibration. Each set of specimens, in the latter case,

was cast at one time as the molds rested on a vibrating table. When casting specimens by vibration, the concrete was placed in the molds by means of a large spoon, filling about one-fifth each time. No additional concrete was added until the matrix of that previously placed was flushed to the surface. The specimens for each complete set, or each investigation, were made at one time by continuous operation. They were covered by a wet cloth and stored at room temperature until the next day, and were then removed to damp closet storage until time for test. Leadite caps (top and bottom) were used when testing all specimens.

The deficiency in fine aggregate of two stone sand mixtures (1-A, Table 3, and BS-1, Table 4) did not permit satisfactory compaction.* The result was that about one-half the vertical surface was honey-combed in each of the specimens. All others were sufficiently workable to permit proper compaction by the method used.

DISCUSSION OF RESULTS

In this discussion the order in which the tabular data are presented will be adhered to as nearly as possible. Inter-comparisons of data from two or more series are shown by a number of diagrams, one or two of which depend upon data omitted from this report for the sake of brevity.

The effect of consistency on compressive strength of concrete

Table 1 contains the summarized data of a series of tests which indicate that consistency varies with change in cement content of similar mixtures and that strength is dependent on consistency apart from water cement ratio, even in concretes suitable for hand placing. As stated in the outline, seven concretes were included which varied in cement content from 0.5 to 2.0 bbl. of cement per cu. yd. Fine and coarse aggregates were constant throughout, but minus 100 mesh dolomite was added in such quantity as to keep the cement plus dolomite at the equivalent of 2.0 bbl. of cement per cu. yd. on an absolute volume basis. In the seven concretes (specimens placed by hand rodding) the slump was kept as nearly as possible at 6 in.

Three additional concretes were also included as a variation of Group D, all containing 1.25 bbl. of cement per cu. yd., but gaged to 4-, 2- and 0-in. slump, respectively. The reduced water content in this group permitted a comparison of compressive strengths of concretes having the same W/C but different consistencies.

The 3-months compressive strengths from Table 1 are plotted against the total water content of the concretes in Fig. 3. The strengths of the

*The relative workability of these mixtures could be determined without difficulty. The frequency of the vibrator attached to the workability device was 8000 per minute with the vibratory action being transmitted laterally. The vibrating table upon which the compressive test specimens were cast operated at a lower frequency of 3600 per minute, and the vibratory action was transmitted in a vertical direction.

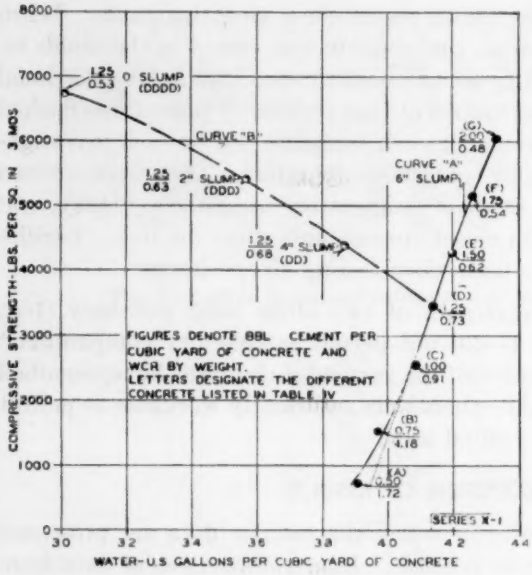

COMPRESSIVE STRENGTH-LBS. PER SQ IN 3 MOS
WATER-U.S GALLONS PER CUBIC YARD OF CONCRETE
1.25 0.53 0 SLUMP (DDDD)
CURVE "B"
1.25 0.63 2" SLUMP (DDD)
1.25 0.68 4" SLUMP (DD)
CURVE "A" 6" SLUMP
(G) 2.00 0.48
(F) 1.75 0.54
(E) 1.50 0.62
(D) 1.25 0.73
(C) 1.00 0.91
(B) 0.75 4.18
(A) 0.50 1.72
FIGURES DENOTE BBL CEMENT PER CUBIC YARD OF CONCRETE AND WCR BY WEIGHT.
LETTERS DESIGNATE THE DIFFERENT CONCRETE LISTED IN TABLE IV
SERIES X-1

Fig. 3 — Relation between compressive strength of concrete and water content, water - cement ratio and consistency.

6-in. slump concretes lie in the full curve, which is so nearly s straight line that one might be justified in making a slight correction (as shown by the dotted line) to compensate for the lower slumps of the two weakest mixtures, A and B. In the same manner, the strengths of the D groups are plotted on the dash line.

Curve A of Fig. 3 is significant in showing that change in the cement content with the minus 100 mesh aggregate compensation requires a change in water content if equal consistency is to be maintained. This is contrary to the commonly accepted view that slump and total water per unit volume remain constant under these conditions. The change in water content for constant consistency, while slight for small changes in cement content, is nevertheless proportional to the quantity of cement in the mix and amounts to over 10 per cent for the range covered in this series. In the case of the mixtures shown by Curve A, Fig. 3, the additional water required to maintain the given slump was directly proportional to the increase in cement content (and decrease in the minus 100 dolomitic aggregate). The mixtures in the series shown by Fig. 8, however, indicate the mixture containing the intermediate cement content (1.25 bbl. per cu. yd.) as having the lowest water content for a given placeability. A greater amount of water was necessary for equal placeability as the mixture became either leaner or richer in cement.

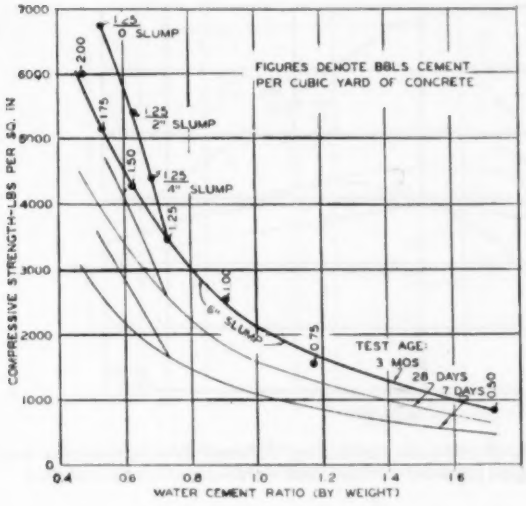


Fig. 4 — Relation between compressive strength of concrete and water - cement ratio, cement content and consistency.

The seeming inconsistency between these two series is clearly explained by Dunagan.*

Curve A in Fig. 3 shows that for constant slump, the strength is an inverse function of the water cement ratio, but Curve B shows that when cement content is constant and slump is reduced, the strengths increase above that indicated by the water cement ratio, and therefore the strength must depend to some extent upon consistency or relative water content.

The discrepancy is seen more easily in Fig. 4, in which the data are plotted in the conventional method — strength vs. W/C. There it is seen that a compressive strength of 5000 psi. at 3 months is obtained with 1.25 bbl. of cement per cu. yd. at approximately 3-in. slump (W/C = 0.65), or with 1.7 bbl. of cement per cu. yd. at 6-in. slump (W/C = 0.54). Similar differences occur in the strength relations at 7 and 28 days. It is obvious that greater economy will result in the use of drier consistencies and placement by vibration than with wetter consistencies by hand methods, and that the factor governing strength is not water-cement ratio alone. A leaner mixture, at a higher water-cement ratio, produces a compressive strength equal to that of a richer mixture at a lower water-cement ratio.

Effect of grading of aggregate on compressive strength of concrete at vibrating consistency

Data from Table 2 are plotted in Fig. 5 — compressive strength vs. fineness modulus of aggregate. The concretes were made with five

*W. M. Dunagan: The Application of Some of the Newer Concepts to the Design of Concrete Mixes, ACI JOURNAL, June, 1940; *Proceedings* V. 36. See (b) The validity of the Slater-Lyse rule, p. 667.

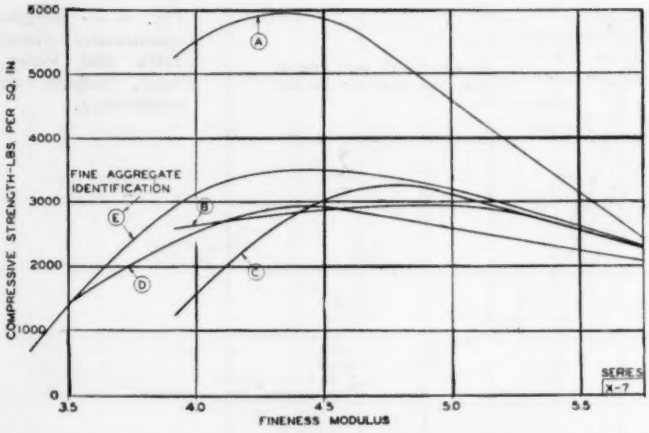


Fig. 5—Relation between strength of concrete containing different fine aggregates and fineness modulus of total aggregate; cement content and water-cement ratio constant

different sands used in varying proportions with the coarse aggregate, and all mixtures were of the same cement content and water-cement ratio. Each value of strength in Table 2 was obtained from the average of 10 specimens. The curves in Fig. 5 indicate that considerable variation in the strength of vibrated concrete can exist as the result of changes in type and gradation of aggregate. The outstanding quality of sand A is shown, but for a more complete and fair comparison, some adjustment would have to be made on the basis of comparative workability—not possible by present accepted tests in this type of concrete.

Relation between placeability of stiff concretes and character of aggregate

The introduction of a method for measuring the relative placeability of low or no slump concretes opens up new possibilities in design and control. Table 3 summarizes the data of a series of concretes, in which two types of stone sand were used, one being the re-shaped sand A, and the other sand F, composed of relatively sharp and elongated particles from the primary crusher. Grading of the total aggregate was changed as before, by changing the ratio of fine to coarse, and the placeability was determined by the method previously referred to under "Outline of Tests." The same cement content was used throughout, but a higher water ratio was required for sand F than for sand A to give about the same optimum placeability. The results are plotted in Fig. 6. Both placeability, which is indicated as the time in seconds required for maximum compaction, and compressive strength, are plotted against fineness modulus of aggregate. Maximum placeability is reached at the lowest points on the dot-dash curves. It is seen that the compressive strength falls off rapidly for gradations coarser than that giving maximum placeability.

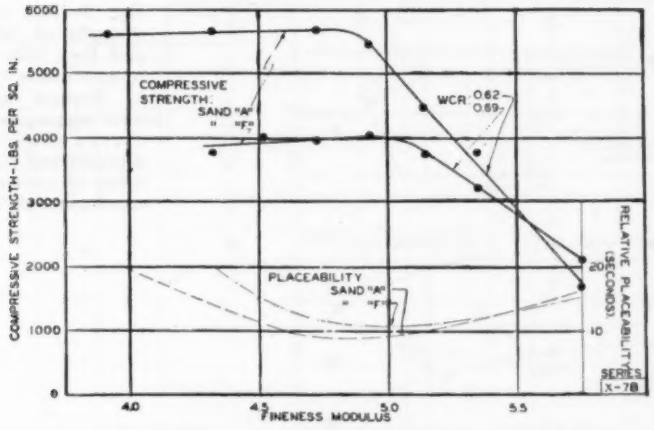


Fig. 6—Strength and placeability of concrete containing stone sands A and F in relation to fineness modulus of total aggregate.

The superiority of Sand A is indicated by a reduction in W/C of about 10 per cent, and an increase in strength of at least 40 per cent over that of sand F. Further examination shows that while maximum placeability and strengths are consistent for each of the sands, the maxima for sand A occur at a somewhat lower fineness modulus than do those for sand F, and at still coarser grading the curves cross. The significance of this relation may lie in the fact that many more particles of the sharp and splintery sand F are found in a given sieve fraction than for sand A, indicating that on a basis of equal surface the curves for sand F should lie farther to the left. This would tend to reconcile maximum workability with maximum strength, regardless of particle shape, and would eliminate the crossing of the curves, at least for any grading that could be called workable.

Placeability and strength of stiff concretes in relation to grading of aggregate and richness of mix

Table 4 gives further data on sand A when used in lean, medium and rich mixtures. For each cement content a group of tests was made similar to those in the two series just described, that is, with varying proportions of fine and coarse aggregate, all having the same W/C and cement content. The compressive strengths are plotted in the full curves of Fig. 7, which show that the strengths fall rather rapidly beyond the gradation which gives maximum placeability, indicated by the peaks of the dot-dash curves. It is important to note that the points of maximum placeability are not much affected by cement content or water-cement ratio, but seem to be tied up to fineness modulus of the total

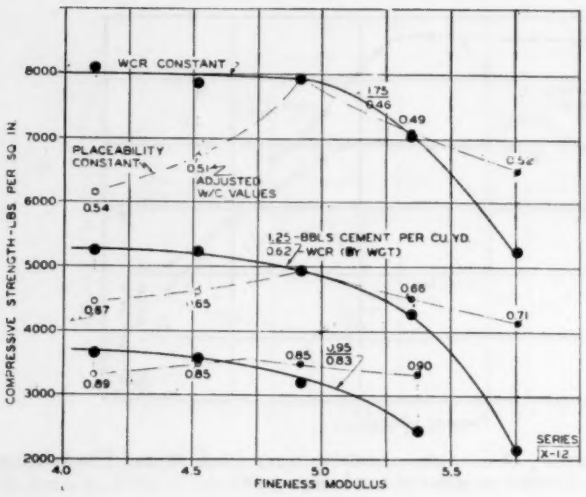


Fig. 7—Full curves show strengths of rich, medium and lean concretes, each at constant w/c in relation to fineness modulus of total aggregates. Dotted curves show adjusted strengths and water-cement ratios of concretes having equal placeability.

aggregate. This suggests the idea of an optimum gradation of aggregates for this type of concrete, short of which placeability would be penalized, and beyond which loss of placeability would be accompanied by very serious losses of strength.

In Fig. 7, let us focus attention on the dot-dash curves labeled "Placeability Constant." To understand the construction, see Fig. 8, in which the placeabilities taken from Table 4 are plotted against fineness modulus of aggregate. Maximum placeability corresponds to the lowest points on these curves, which are the peaks of the constant placeability curves in Fig. 7. In Fig. 8 it is obvious that constant placeability curves are horizontal lines, three of which are drawn tangent to the full curves at their minimum points. Points to the right or left of these minima on the

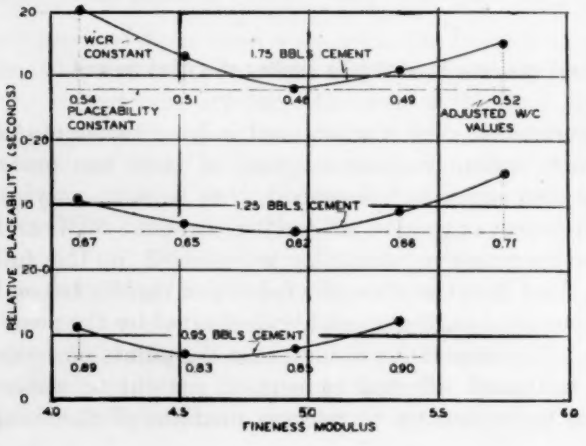


Fig. 8—Placeability curves of rich, medium and lean concretes, each at constant w/c, in relation to fineness modulus of total aggregate. Figures are the adjusted w/c values necessary to bring the concretes to constant placeability.

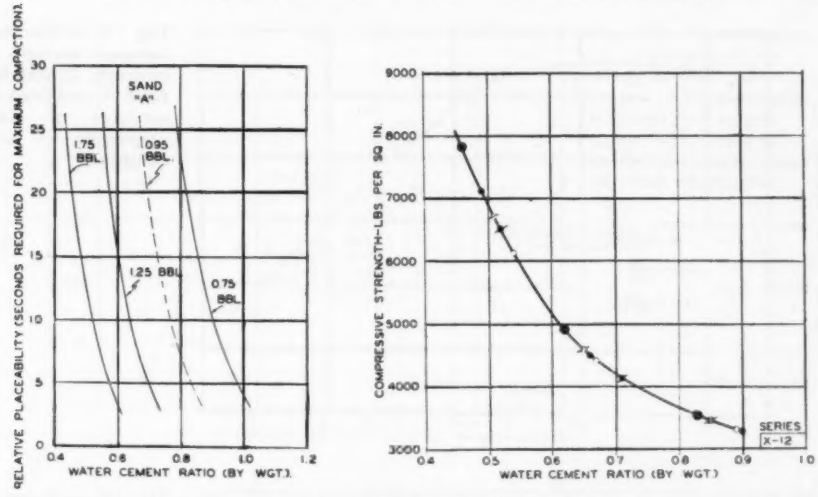


Fig. 9—Curves show relation between placeability of rich, medium and lean concretes and water-cement ratio.

Fig. 10—Strength—w/c curve for determination of adjusted strength values in Fig. 7.

full curves correspond to less workable mixtures, that is, more water would have to be added to bring them to the maximum placeability. The increased water-cement ratio necessary to give this increased workability is indicated below each of the original points of the full curves. These derived W/C values are taken from Fig. 9, which shows the relation between placeability and W/C for the mixtures listed in Table 4. Finally the strengths corresponding to the adjusted W/C values in Fig. 8 are interpolated from the curve in Fig. 10, which shows the Strength — W/C curve drawn through the three points (the larger solid circles) corresponding to the mixtures of maximum placeability in Table 4.

The strengths determined in the above manner locate the points for the constant placeability curves in Fig. 7. It will be noted that the strength of the concrete, when adjusted for equal placeability, (workability) decreases progressively as the gradation of the aggregate becomes finer or coarser than the optimum. This relative decrease is greater as the cement content of the mixture is increased. Also in the lean mixtures (0.95 bbl. per cu. yd.) the fineness modulus of aggregate corresponding to maximum placeability is lower than that for the two richer mixtures.

It can be noted that the definite tendency to establish optimum gradations of aggregate, measured by relative placeability, is substantiated in general by Dunagan, in using Powers'* "Remolding effort" test.

The lack of better coincidence between the actual strengths and the estimated strengths of the coarser gradations in Fig. 7 is attributed to

*T. C. Powers: Some Workability Studies. ACI JOURNAL, Feb. 1932, *Proceedings* V. 28.

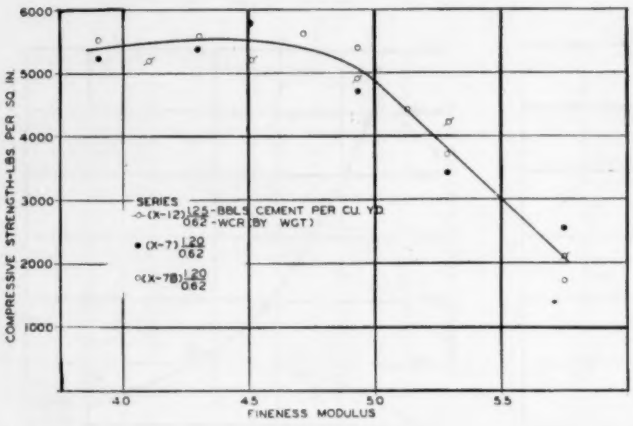


Fig. 11 — Relation between strength of concrete containing sand A and fineness modulus of total aggregate — w/c constant.

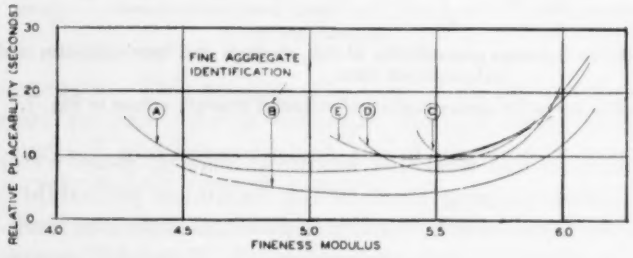


Fig. 12 — Relation between placeability of concretes containing different fine aggregates and fineness modulus of total aggregates. Cement content and water-cement ratio constant.

the fact that the concrete is too harsh to remain homogeneous and that something resembling particle interference is present. It is seen that the divergence between the full curves and the dot-dash curves increases as the mixes are leaner; it is also more pronounced as the aggregate is coarser. The practical effect upon the placing of the concrete in the molds was that in the 0.95 bbl. mixture with aggregate of fineness modulus 5.76, compaction of the specimens was not possible. In such mixtures the water-cement ratio is meaningless so far as strength is concerned.

Importance of the placeability test in indicating optimum gradation

Fig. 11 shows the strengths of concretes made with sand A, as taken from Tables 2, 3 and 4, for a water-cement ratio of 0.62. These are from different series made at different times — nevertheless the curve is a fair locus showing strength change with change in the fineness modulus of the total aggregate. Two points may be emphasized: (1) That the maximum placeability, as shown in Fig. 6 and 7, gives about the maximum strength; and (2) that the water-cement ratio does not appear to be a dependable index of strength when the aggregate grading is coarser than that giving maximum placeability. The author believes it important to realize that without the placeability test, or some means of indi-

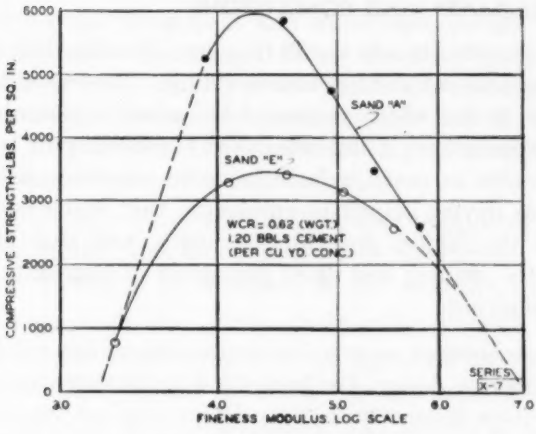


Fig. 13—Variation in compressive strengths of concretes containing different fine aggregates with change in fineness modulus of total aggregate.

cating the relative workability of stiff concretes, the designer's judgment as to the proper proportions of fine and coarse aggregate would in the majority of cases prescribe a combination too coarse to give maximum placeability and strength obtainable with the water-cement ratio used.

Studies of natural sands

The tests reported here have related mostly to concrete mixtures containing dolomitic stone sand as fine aggregate. However, four of the compressive strength curves shown in Fig. 5 (data from Table 2) are for concrete containing natural sands. Table 5 contains further data on the comparative placeabilities of concretes made with these five sands, the coarse aggregates in this case being crushed dolomite, $\frac{3}{8}$ to $1\frac{1}{2}$ in. The placeability curves for these mixtures are shown in Fig. 12. Bearing in mind that all these concretes have the same low cement content and the same water-cement ratio, quite some differences are shown in the effects of the different sands. The data indicate that the combined effect of particle shape and surface texture have a considerable influence on the workability of concrete of this character. It is probable that in richer mixtures the properties of the sands which affect workability as here shown would have less relative significance.

The effect of grading of the total aggregate on the compressive strength of concrete, when the former is varied over the entire practicable range, is shown in Fig. 13. The data are for concretes containing sands A and E, Table 2. The appearance of the curves suggests that the variation in strength due to change in grading is somewhat proportional to the quality of the concrete produced. It would follow that as quality is enhanced by a superior aggregate, it would be necessary to maintain closer control over proportioning for maximum benefit.

SUMMARY AND CONCLUSIONS

Most of the studies reported herein would be generally related to mass concretes, of low cement content and low to zero slump. They were aided very materially by the device which measured the relative placeability of low and no-slump concretes. It indicates that consistency or workability in this region varies as much, relatively, as in concretes suitable for hand placing. This device helped to emphasize that water-cement ratio was only one of the factors governing strength, and that placeability as affected by the grading and other properties of aggregate was possibly even more significant.

While this effect of consistency or relative water content has not been widely recognized in concrete design, the Series X-1, involving concretes of zero to 6-in. slump, show distinctly that consistency has an important effect upon strength, and that the effect is more marked as the slump is reduced. Carried on into the region of zero slump, consistency, or perhaps we should now say, placeability, has almost a predominating influence, as many of the diagrams show. The experience suggests the possibility of refinements in mix design, particularly for concrete of vibrating consistency.

Another factor is that smooth and rounded aggregate in low slump concrete has no advantage that can be attributed to lower water requirement for given workability, over rough, angular material, without proper consideration of the relative structural qualities (strength). Many times the latter exhibits higher compressive strength, even though higher water ratio is required to provide equal workability.

Studies of this nature also invite new consideration of an optimum grading of aggregate — possibly of lower maximum size and higher in fines than orthodox theory has demanded. Placeability would be enhanced by reducing the average diameter of the aggregate particles. Void spaces in the aggregate prior to compaction would tend to be smaller and more uniformly distributed. Vibratory force for compaction should act more quickly. The films of paste between the aggregates would be thinner, and, in the opinion of many, form a more efficient binder.

If one can accept the idea that greater efficiency of cement and higher quality of concrete lie in the drier consistencies, and that aggregate of the lowest surface area, and the lowest water cement ratios, are not always criteria of the best concrete, he will have the gist of this paper.

The following conclusions are offered as a basis for further study:

1. For each aggregate or set of aggregates of given maximum size, an optimum gradation exists which will provide highest workability in

concrete when the cement and water contents are constant. Departure from this optimum toward either the finer or coarser gradations will require a greater quantity of mixing water to maintain equal workability. Such increased mixing water will result in loss of strength. With aggregate of uniform quality, the gradation producing maximum workability will also produce maximum strength.

2. The total water content, per unit volume of concrete, required to provide equal consistency with given aggregates varies with the paste content and must be increased as the mixtures become richer.

3. The consistency of the mixture has an effect upon the compressive strength of concrete which is not governed by the water cement ratio when used to designate all relative proportions of water and cement.

4. An appreciable difference in the concrete making qualities of aggregate can exist. It would appear that these qualities are developed to a greater extent (within limits) as the general quality of the concrete is improved.

Grateful acknowledgment is made to J. C. Pearson, Director of Research, Lehigh Portland Cement Co., for his assistance in reviewing this paper, and for supervising the freezing and thawing tests reported in the Appendix. The freezing and thawing test program was made possible through the courtesy and facilities of the Lehigh company.

APPENDIX

Series X-1 of the foregoing paper included sufficient 3 x 6-in. cylinders to permit freezing and thawing tests to be made of five specimens of each mixture. The age of these specimens at the beginning of the test was three months. They were prepared for test by sawing one inch from each end and then treating the vertical surface by rolling in acid and brushing until all cement skin was removed. They were soaked in water for four days and then weighed in saturated surface dry condition. These weights were taken as the initial weights of the specimens. Subsequent weighings were made in similar manner, that is, in saturated surface dry condition, and the losses in weight from frost action were taken as the differences in these saturated surface dry weights.

The specimens were frozen daily in air (in saturated condition) at 0 F. to minus 5 F. from about 9:30 a.m. to 3:30 p.m. and again during the night. They were thawed in water at room temperature in the early morning and again in the late afternoon. Normally 11 cycles of freezing and thawing were secured per week, but this was frequently altered by shutdowns, holidays, etc.

Regular weekly weighings of the cylinders were not started until visible losses were observed. In many cases the losses became irregular as the cylinders approached failure, and it was difficult to assign failure to a definite number of cycles.

The wide range in losses of weight for a given number of cycles in the stronger groups suggests that the small cylinders are susceptible to variations in casting, etc., which cause relatively large differences in apparent durability of similar specimens. Losses

of the order of 10 per cent were recorded in some cases with no indication of disintegration. On the other hand, there were suggestions of breakdown after losses of 2 or 3 per cent up to 15 per cent or more.

In general, the condition of the cylinders was checked by weekly weighings and by photographs until complete breakdown occurred or up to the end of the test program, which was stopped at 850 cycles.

Many specimens of the stronger groups did not register complete failure at the end of the program. The strongest group (DDDD) lost but an average of 3.84 per cent during the entire period. All specimens of the group containing the leanest mixture (A) disintegrated between the 38th and 48th cycles.

Discussion

Table A gives essentials for understanding the analysis of the test results:

TABLE A

Mix	Cycles at 100 g. Loss						C/W, by Wt.	Bbl. Cement per Cu.Yd.	Slump In.	3 Mo. Compr. Str., p.s.i.
	1	2	3	4	5	Avg.				
A	21	29	21	34	28	27	.58	.50	5½	830
B	69	60	57	62	65	63	.86	.75	5¾	1550
C	102	113	408	114	247	197	1.10	1.00	6	2560
D	176	209	533	(890)	252	412	1.37	1.25	6	3480
DD	207	(1080)	410	426	617	548	1.47	1.25	4	4400
DDD	270	(1350)	443	714	(930)	741	1.58	1.25	2	5410
DDDD	(1190)	(1530)	(1200)	(1480)	(1610)	1402	1.88	1.25	0	6730
E	392	639	388	(1350)	557	665	1.61	1.50	6	4290
F	425	807	(980)	(1010)	(1150)	874	1.85	1.75	6	5160
G	552	622	(1500)	(1610)	(1520)	(1155)	2.08	2.00	6	6040

As previously mentioned, a considerable number of the cylinders survived the 850 cycles of freezing and thawing. In order to get a numerical rating for the life of the specimens, an arbitrary scheme was adopted. An end point of 6 per cent loss (70 grams) obviously penalized the more resistant groups. A 10 per cent loss (115 grams) was somewhat greater than the maximum loss of any of the specimens which did not fail in 850 cycles. This end point was quite promising, especially when the surviving cylinders were rated by an amount proportional to the difference between 115 grams and their actual losses. However, an estimate of durability on this basis involved such a degree of extrapolation for the loss curves of the highly resistant DDDD group that it seemed best to drop back to a more conservative basis and a 100 gram loss was finally adopted as the end point. There were three cylinders: DD-5, DDD-4, and F-2, which ran about 100 grams loss without signs of approaching failure, and these were definitely penalized by the scheme. For the remaining sixteen cylinders, life-cycles were calculated simply by adding 10 cycles for each gram loss short of 100. This would penalize some, and perhaps favor some of the most resistant, but it seems to be about as fair an estimate as could be made from the data.

The life-cycles given in parentheses in Table 4 were derived in this manner. The average number of cycles for each group, as given in the seventh column of the table, were then taken as representing the durability of the specimens. These values, together with the 3 mo. compressive strengths, are plotted against C/W values in Fig. A. They could have been plotted vs. W/C values, as in Fig. 4 of the paper, but the divergence of the two branches of the curves is more readily appreciated in the C/W diagram.

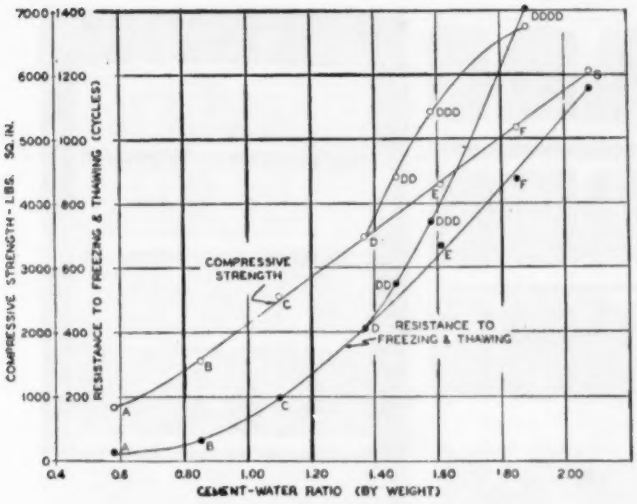


Fig. A — Relation between compressive strength and durability of concrete and the cement - water ratio, cement content and consistency.

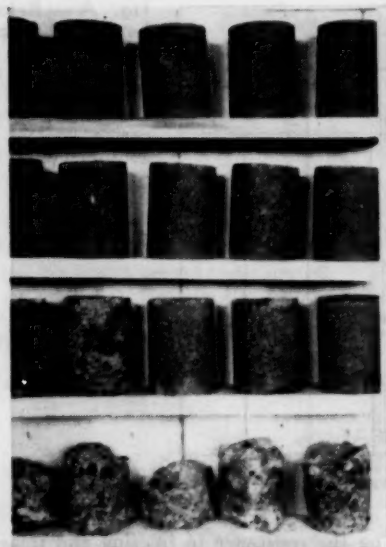
Keeping in mind that the method of rating the resistance to freezing and thawing can at best give but probable values, it is interesting to observe that the strength and durability curves have similar characteristics. In other words, durability, like strength, seems to be markedly improved by drier consistencies, regardless of cement content and cement water ratio. Fig. A also shows that the slopes of the two branches of the durability curve are somewhat greater than those of the two branches of the strength curve, which may be taken as an indication that durability is benefitted to an even greater degree than strength by the use of the drier consistencies.

Whether one considers that the foregoing scheme of analysis is justified or not, there is no question that the DDDD group of cylinders at the end of the test had suffered less damage than any other group — even in the G group two of the cylinders had failed. This means that the concrete containing 1.25 bbl. of cement per cubic yard with a water-cement ratio of .53 and a slump of zero was definitely more resistant to freezing and thawing than the concrete containing 2 bbl. of cement per cubic yard with a water-cement ratio of .48 and a slump of 6 in.

Fig. B, C and D are offered as assistance to the reader if attempt is made to understand the broad interpretation placed upon these test results. Fig. B is shown mainly to afford opportunity to visualize the condition of Group A at the weighing immediately prior to failure. Fig. C shows the condition of the D groups at the termination of the program. This figure presents a picture which shows not only the excellent condition of the DDDD group, but also gives evidence of the wide spread in the individual test values. In the DD and DDD sets, three and two specimens, respectively, have been completely disintegrated, which resulted in their removal from the picture. The remaining specimens show but slight loss. Fig. D shows the condition of the E, F, G groups at the end of 850 cycles. Group G registers complete disintegration of two specimens with the remaining three practically intact.

Conclusion

The analysis of these results is intended to serve two purposes. The first is to indicate that the resistance to freezing is influenced by the water-cement ratio. In accordance with this relation, slight durability can be noted for the concrete of low cement



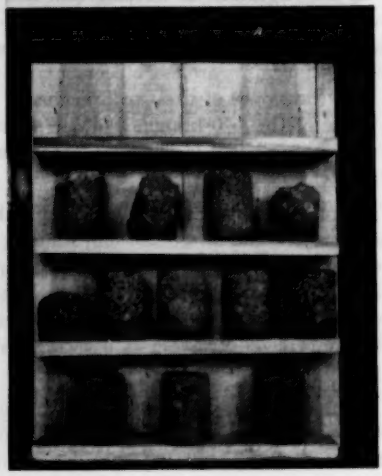


Fig. B (upper left)—Condition of groups A, B, C, D, (noted from bottom to top) at end of 38 cycles at freezing and thawing.

Fig. C (upper right)—Condition of groups D, DD, DDD, DDDD, (noted from top to bottom) at end of 850 cycles of freezing and thawing.

Fig. D (lower left)—Condition of groups E, F, G, (noted from top to bottom) at end of 850 cycles of freezing and thawing.

content which possesses relatively high W/C. If the analysis is accepted, it is obvious that, regardless of compressive strength, the minimum cement content for exposed concrete should be based upon durability rather than minimum compressive strength.

The other purpose is to call attention to the data which are interpreted to show that evaluation of the respective losses in the stronger mixtures tends strongly to substantiate the same effect of consistency upon quality as indicated by the compressive tests of similar mixtures. They indicate that the consistency has an effect upon concrete quality which is not taken into account in the orthodox method of water-cement ratio application.

Discussion of this paper should reach the ACI Secretary in triplicate by Mar. 1, 1942, for publication in the JOURNAL for June, 1942

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Discussion of a paper by G. W. Hutchinson:

The Influence of Vibration, Consistency, and Grading of Aggregate Upon the Design of Concrete*

DISCUSSION BY C. H. FORESMAN, C. A. G. WEYMOUTH and MR. HUTCHINSON

By C. H. FORESMAN†

Mr. Hutchinson's paper presents some interesting and informative data. The conclusions are properly his own but it occurs to me that a discussion, based on investigations of mine, might throw a different light on his interpretation of the results.

In connection with the water-cement ratio, it would be well to remember that it does not furnish any absolute values for the strength of concrete. The W/C-strength relationship is definitely qualified by the phrase "other conditions being the same". Any change in any of the ingredients other than the water and the cement will set up a new W/C-strength relationship. In Series X-1, Table 1, Mr. Hutchinson has made such a change in each of the mixes by changing the amount of fine dolomite.

Mr. Hutchinson does not furnish any information about the fine dolomite other than that it passes a 100-mesh sieve. Since he has used it in the nature of a blend with cement rather than as an aggregate, it may be assumed that it was at least as fine as the cement. Much of the strength of concrete comes from the contact of cement crystals formed during hydration. The crystals and gel bond with each other and with particles of aggregate or other inert material in the mix. Water in excess of the amount necessary to hydrate the cement reduces the strength because it separates the crystals. When inert material finer than cement is added to the mix it tends to do two things: (1) fill the natural voids in the cement or between the cement particles (thus providing additional contact) and (2) decreases the amount of water per unit volume of concrete necessary to maintain a given degree of workability. The latter

*ACI JOURNAL, Sept. 1941; *Proceedings* V. 38, p. 9.
†Concrete Engineer, Board of Public Education, Pittsburgh, Pa.

is accomplished by displacement of some of the void filling water, allowing a greater percentage of the total water to be used for lubrication or mobility. This is true up to an optimum ratio between the dolomite and the cement. If the ratio is further increased it may be necessary to provide additional water to fill the voids in the dolomite. Certainly there will be an increasing amount of contact between dolomite and dolomite which, because of the absence of cementitious value would result in decreased strength. If the structural value of the flour is good; if it bonds well with cement particles; if the grading of the flour is such that the voids in the cement range will be reduced without contact of non-cementitious particles; or if the flour is cementitious in itself there may be no reduction in strength. Some materials may even show an increase in strength at the proper ratios between the flour and the cement. In most cases, however, some reduction in strength may be expected but generally the total amount of water necessary to maintain a given slump is reduced. (See Table 1)

In the matter of gradation of aggregates, Mr. Hutchinson has, I believe, correctly interpreted the results of the tests as far as they go. His tests, however, are not sufficiently extensive to cover the full range of possibilities. He has used five sands of different grading (and a sixth of the same grading as one of the others but with differently shaped particles). With a fixed amount of cement paste of a definite water-cement ratio and using placeability as a measure he determines that there is an optimum combination of each one of these sands and a more coarse material of fixed grading. In that he is correct.

My own investigations were made for the purpose of obtaining a gradation of aggregates that would produce a concrete of given characteristics (strength being only one of the qualities) with a minimum cement content. It is natural that, as far as the investigations were similar, the results would be the same. I believe, however, that my investigations were in some respects more extensive than his and that a brief statement of my conclusions might be helpful in the interpretation of his results.

The details of my investigation may well be omitted but the conclusion reached is that there exists not only an optimum relation between the separated sizes of aggregate, but that there is also a maximum relation—one that will produce a maximum absolute volume of aggregate. I have expressed this relation as a Grading Ratio (between one separated size and the separated size next above). The maximum Grading Ratio is seldom practicable in actual construction. It is useful, however, in determining variations between the various sizes. Reduction in grading ratio because of a reduction in absolute volume (increase in

voids) increases the amount of cement paste necessary to embed all of the aggregate particles, but it also increases workability in the sense that the concrete is more easily handled and there is less tendency for the coarse particles to segregate while the concrete is being placed. It follows that the optimum grading ratio is a balance between workability and economy in materials.

A mix of better body and greater economy in cement will be obtained if the proper proportions of all the sizes from the cement range to the maximum size suitable for the work in hand are used. Now, however, we hear much about so-called "gap" gradings. Gap gradings probably originate in the lack of certain sizes of aggregate. Gap grading ratios are as readily obtainable as full grading ratios and, of course, may be used when expedient. Mr. Hutchinson uses such a grading in his tests.

Mr. Hutchinson uses workability to determine the proper amount of $\frac{3}{8}$ - $\frac{3}{4}$-in. material to be used. It will be noted that maximum placeability occurs at different combinations with the different sands. These sands have different gradings and it is natural that the best combinations with a larger size aggregate would not be the same. Of particular interest is Table 3 where he used sands of the same grading but with particles of a different shape. The shape of the particles affects the grading ratio, hence it would be expected that the optimum combination with the larger sizes would be different.

In this connection, Mr. Hutchinson mentions "what would ordinarily be regarded as over-sanded mixtures". Because of an arbitrary separation of aggregates at about a No. 4 sieve, it has become customary to regard the materials above and below this sieve as entirely separate and different entities. The large and small particles may differ in many respects but so also do the particles within the range of what is called one aggregate (fine or coarse). There is, in actuality, no such thing as an over- or under-sanded mix; there is merely an excess or deficiency of certain sizes within the total range. Much of the difficulty in comprehending the proper design of a concrete mix would be eliminated if the "fine and coarse aggregate" fixation were removed.

Mr. Hutchinson in Fig. 5 plots from data in Table 2 strengths against fineness modulus. His charts show the strength is not dependent on either a high or the same fineness modulus. The answer is that the fineness modulus is not a good measure of the grading of an aggregate. It grossly exaggerates the larger sizes. To be properly used, there should be a series of fineness moduli—one for the material below each sieve. When this is done the fineness modulus becomes a jumble of multiplications of the percentages of material between the various sieves. The simple thing to do is to examine the grading by individual sizes, bearing in mind that here does exist a proper ratio between them.

With reference to the compressive strengths obtained, Mr. Hutchinson has not stated the absolute volume (percentage of voids) of the combined aggregates. It is likely, however, that in some of the mixes the actual volume of voids exceeds the volume of cement paste. Lack of sufficient paste to fill the voids in the aggregate would act the same as an excess of water—it would prevent contact (bond) between the cement particles or between the cement and aggregate. The strength of the concrete will follow the absolute volume of the aggregates up to the point where the volume of aggregate voids equals the volume of the cement paste. Beyond that point, the strength, except as influenced by the structural value of the aggregate particles and by other and minor characteristics of the aggregate, will remain constant as long as the water-cement ratio is constant.

I am deeply indebted to Mr. Hutchinson for this series of tests since it furnishes independent confirmation of the conclusions I drew from my own investigations. I would be glad to know if this discussion has in any way influenced his interpretation of his own results.

By C. A. G. WEYMOUTH*

This paper is one of the most important contributions, in recent times, of test data on concrete mixes. It provides accurate information for checking theories of concrete mix design, but in view of the amount of work involved, Mr. Hutchinson wisely leaves this correlation to those especially interested in such theories. The writer finds that these data provide a striking confirmation of his own general theory of mix design and establish definite values for some of the criteria for jump grading which has been questioned.

This method makes use of the "relative density of the size-group" which permits the calculation of the average spacing in the mix of the particles of any diameter. The theory for workable mixtures requires that the clear distance between particles of any size shall be equal to, or larger than, the particles of the next smaller size which are present in *appreciable* quantity, and that a smaller clear distance causes particle interference and unworkability. Values of the relative density computed by the writer's formula for this critical clear distance are given in Table D. Another phase of the theory involves the size of the void pocket of the size-group as a criterion for plastic suspension of the aggregate in the mix. Doubt has been expressed by some writers of the adequacy of these criteria as applying equally to conditions of continuous, jump, or semi-jump gradings. It has been thought, for example, that the critical magnitude of the relative density should vary with the

*Raymond G. Osborne, Bureau of Tests and Inspection, Los Angeles.

amount of material in the size-group having the next smaller diameters. It will be seen, however, that these data provide a satisfactory solution.

Since the fineness modulus is given in most of the tables and figures of the paper, the writer's first study was made to see how well these data fulfilled Duff Abrams' theory of mix design. For this purpose those mixes which have the shortest placing time or indicate a critical compressive strength are interpolated and listed in Table A. Values for Abrams' "mix" are estimated from the cement contents and the F.M.-values are taken from Table 3 of his original Bulletin No. 1, "Design of Concrete Mixtures." Upon comparing the fineness moduli obtained by experiment with the tabular values, it is seen that in some of the mixes the agreement is remarkably close. In general the best correlation is given with those aggregates which have the material graded evenly in the largest number of size-groups. Sands C, D, and E, and the ¾-in. stone, all have large concentrations in one or two groups and their concretes provide poor agreement with Abrams' table. To find reasons for this disagreement, one must search further into Abrams' fundamental ideal for workable mixes: "the aggregate grading should not be too coarse for its maximum size and for the amount of cement used." Abrams recognized that the fineness modulus of the combined aggregate is too broad a function to make fine distinctions of whether some parts of the grading might be too coarse for its matrix, and he sets up the requirement that the sand portion of the mix should not be too coarse for best results in concrete mixtures: the sand-cement ratio must also comply with the table for fineness and maximum size of the sand.

The writer accepts Abrams' ideal for workable mixes by defining "too coarse" as a condition applying to *any* size-group of the combined grading which is concentrated in such amount that it causes particle interference with its matrix. In place of fineness modulus he uses the relative density of the size-group to limit this concentration. When any size is crowded into a mix until the pieces are too close to permit free movement of its matrix as the concrete is placed, some of the finer elements become separated from the coarse, a form of internal segregation which increases the voids in the paste. The amount of cement used must be thinned with additional water or take on air to fill the void structure of the segregated portions and a decided loss of plastic quality results.

A study of a typical series of concretes analyzed by the relative density method is given in Table B. The values of this function exceeding the limits given in Table D are italicized to indicate the presence of particle interference. In the undersanded mixes the ¾ to 1½-in. group dominates the mix while the No. 30 to 16 group dominates in the

oversanded concretes. It will be seen that particle interference decreases from both sources as the proportions approach those of Mix 4 which has the least combined excess relative density. In the example no proportion entirely eliminates particle interference. Considered as a

TABLE A—FINENESS MODULUS THEORY

Nos.	Sand	Max. Size in.	Bbl. per Yard	W/C by Wt.	Duff Abrams		F. M. from Test Data		
					"Mix"	F. M.	Maximum Strength	Maximum Workability	Placeability Seconds
1	A	1½	1.00	0.70	6.7	5.08	....	5.10	6.5
2	B	1½	1.00	0.70	6.7	5.33	....	5.31	4.0
3	C	1½	1.00	0.70	6.7	5.33	....	5.59	9.5
4	D	1½	1.00	0.70	6.7	5.33	....	5.52	7.7
5	E	1½	1.00	0.70	6.7	5.33	....	5.42	9.6
6	A	¾	0.95	0.83	7.1	4.25	4.3	4.65	6.5
7	A	¾	1.20	0.62	5.6	4.41	4.73	4.83	9.0
8	A	¾	1.25	0.62	5.3	4.46	4.52	4.87	6.0
9	A	¾	1.75	0.46	3.5	4.85	4.9	4.97	8.0
10	B	¾	1.20	0.62	5.6	4.66	4.91	....	....
11	F	¾	1.20	0.69	5.6	4.10	4.94	4.94	11.0
12 & 15	C	¾	1.20	0.62	5.6	4.66	4.80	5.46*	....
13 & 16	D	¾	1.20	0.62	5.6	4.66	4.42	5.43*	....
14 & 17	E	¾	1.20	0.62	5.6	4.66	4.43	5.31*	....

Numbers refer to those listed in Table C.

*These values estimated for optimum sand percentage judged by relative densities of the dominant size-groups of sand and stone.

"Mix" estimated by means of Fig. 6 in "Design and Control of Concrete Mixtures," P.C.A. Sept. 1924. Abrams' F.M. are interpolated from Table 3, "Bulletin 1, Design of Concrete Mixtures" by Duff Abrams, Structural Research Laboratory, Lewis Institute, Chicago, 1918, with indicated corrections for shape of particles.

TABLE B—VARIATION OF RELATIVE DENSITIES

Sand A with ⅜-1½-in. Stone

	fm.	Ratio	a	b	Size-Groups								
					−100	No. 100 to 50	No. 50 to 50	No. 30 to 16	No. 16 to 8	No. 8 to 4	No. 4 to ⅜"	⅜ to ¾"	¾ to 1½"
					Relative Densities								
Sand Grading					15%	9%	12%	21%	23%	20%	0%	50%	50%
Stone Grading													
1	6.11	3 : 7	.232	.542	.134	.074	.090	.136	.129	.100	.000	*.372*	*.271*
2	5.65	4 : 6	.310	.464	.172	.093	.110	.161	.150	.116	.000	*.303*	*.232*
3	5.19	5 : 5	.387	.387	.204	.110	.126	*.182*	.166	.127	.000	.240	*.193*
4	5.10	....	.401	.373	.209	.111	.129	*.185*	.168	.128	.000	.230	*.186*
5	4.70	6 : 4	.464	.310	.234	.124	.142	*.198*	*.180*	.135	.000	.183	.155
6	4.27	7 : 3	.542	.232	.263	.138	.154	*.213*	*.189*	.141	.000	.131	.116

problem in concrete mix design, this can be made to disappear by increasing the cement content, or by changing the grading of both the coarse and the fine aggregates slightly.

The curves of Fig. 12 show this characteristic: a decrease in placeability when the relative densities are increased in either the coarse or the fine aggregates from the low points.

The method of computing relative densities is illustrated in Table F.*

The optimum mixes of all the series are given in Table C. Under the relative density of each dominant size-group is the corresponding limit from Table D and the ratio of the two is shown on the line below. This table tells a plain story of the dependence of the character of a mix upon the concentration of its dominant size-groups, but some features need special comment.

Mixes 1 to 5 prove that the optimum proportions for placeability are obtained when the particle interference of the dominant groups in the fine and coarse aggregates just balance. These mixes could be improved as explained above by eliminating particle interference. When the gradings are such that this can be done, the optimum mixes confirm the tabular values of Table D as the limiting condition. Mixes 7, 10, and 11 give close agreement for continuous grading of the dominant group in the sands, and since the sands balance the stone, the value of about 0.356 must be taken as the limit of the relative density when the group heads a jump grading with a gap of two groups. Evidently a larger amount of stone would cause particle interference. Yet Table D gives 0.407 as the theoretical value for the limit of particle interference. This value requires that the spacing of the particles be even. It is evident, however, that if the suspension of the particles is poor heavy pieces will concentrate unevenly causing local particle interference. In these tests the relative density of the stone must be made small enough so that the pieces after settling will not exceed the concentration of 0.407 at the local points of congestion. Attention has been called to the poor suspensive power of jump gradings in several of the writer's publications, and the need to reduce the limit below that computed by the formula.

These data also provide information on the amount of material which may be included in the gap of a jump grading without causing a change in this classification. There is no material in the No. 4 to $\frac{3}{8}$-in. group and the question arises, how much may remain in the No. 8 to No. 4

*The values computed by the writer are based on the following: The air content is neglected; the specific gravity of the natural sands is taken as 2.65; the shape of the grains is difficult to judge from Fig. 1, and d_0 values are assumed for sand A and reshaped stone,0.580; for Sands B, C, D, and E, 0.650; Sand F, 0.510. All interpolations are made from the curves plotted by the author, except Curve A in Fig. 12; replotting this disclosed that the point for the 5:5 mix should be discarded to give a smooth curve through the other points. Inaccuracies in these assumptions will not change the general picture.

For further information on particle interference see the list of published papers given in the footnotes on pages 296-1 and 296-2, ACI JOURNAL *Supplement*, Sept. 1940, *Proc.* V. 36, in discussion of a previous paper by Mr. Hutchinson.

group and permit the grading to function as a jump with a gap of two groups? Mixes 7 to 11 contain from 12 to 22 per cent in the second smaller size. Mix 6 contains 26 per cent. The latter concrete, however, shows a much smaller limit, 0.325, for its stone balance indicating a single gap in the grading. The writer has heretofore used 20 per cent as the limit of the "inappreciable" amount that may remain in the gap but it is here shown to lie between 22 and 26 per cent.

Attention is next directed to Mixes 1, 6, 8, and 9. In these the No. 30 to No. 16 group is not the one which balances the sand against the stone. It is likely that this balance is due to the material in the No. 200 to No. 100 group. The amounts that come to this group from the fines in the sands and from the coarse portion of the cement cannot be computed from the data given. Undoubtedly some of the sand marked "minus No. 100" passes the No. 200 sieve. In Mixes 6, 8, and 9 the fines from the sand decrease but this is apparently compensated for by the increase in plus No. 200 material as the cement increases from 0.95 to 1.75 bbl. per cu. yd.

The last six mixes of Table C afford interesting data on the relation of compressive strength and particle interference, and illustrate the influence of stratification. Mixes 12 to 14 are interpolated for maximum strength from Fig. 5. Mixes 15 to 17 are estimated to give the optimum sand balance for shortest placing time and the least particle interference in the combined aggregate. The wide difference in fineness moduli of these two sets of optima requires an understanding of the forces of temporary suspension in concretes. Some writers consider this suspension to be almost wholly dependent upon the amount of surface area of the cement in the mix. Yet in many of these series where the cement is constant, there is a wide contrast in suspensive quality.

This writer attributes the property of suspension to a proper structure throughout the mix, in which the void pockets of each size must be small enough to hold their matrix. Not only must the fines in the cement subdivide the water and trap it in compartments small enough to retain most of it, but the sand must also subdivide and trap the cement paste before the mortar can suspend the heavy pieces.*

Sands C, D, and E are all poorly graded for the amount of cement in the concretes. Their fines, which might otherwise form the small pockets needed to trap the paste are entirely insufficient to overcome the disruption of good structure caused by their high degree of particle interference. Consequently the maximum strengths are in mixes with smaller amounts of heavy pieces to be suspended than possible for optimum workability, and with further oversanding the strengths fall off rapidly. In contrast,

*For an outline of the writer's theory of plastic suspension see page 1016 Proc. Vol. 41 A.S.T.M. 1941.

TABLE C—ANALYSIS OF OPTIMUM MIXES

No.	Sand Fig.	F. M. Ratio Bbl.	a b	Size-Groups: Minus No. 100	No. 100 to No. 50	No. 50 to No. 30	No. 30 to No. 16	No. 16 to No. 8	No. 8 to No. 4	No. 4 to ⅜ in.	⅜ in. to ¾ in.	¾ in. to 1½ in.
				Relative Densities								
				Criteria: Maximum Workability								
1	A	5.10	.401	.209	.111	.129	*.185*	.168	.128	.000	.230	*.186*
	12	5.19 : 4.81	.373	?			.172					.172
		1.00					108%					108%
2	B	5.31	.377	.017	.115	.330	*.202*	.134	.070	.000	.248	*.198*
	12	4.74 : 5.26	.397				.192					.172
		1.00					112%					115%
3	C	5.59	.333	.047	.123	*.223*	*.251*	.115	.066	.000	.283	*.220*
	12	4.13 : 5.87	.441			.192	.192					.172
		1.00					131%					128%
4	D	5.52	.307	.100	.172	*.250*	.149	.067	.045	.000	.308	*.233*
	12	3.84 : 6.16	.467			.192						.172
		1.00				130%						136%
5	E	5.42	.310	.066	.220	*.253*	*.202*	.030	.000	.000	.302	*.232*
	12	3.86 : 6.14	.464			.192	.192					.172
		1.00				132%						135%

Gaps in the Grading are indicated by underscoring.
Relative Densities, critical or greater, are italicized.
The first set of figures under each are limits given in Table D while the second set represents the percentage of particle interference caused by too much material in the size-group.

Table continued pages 28-10 and 28-11

Table C—continued

No.	Sand Fig.	F. M. Ratio Bbl.	a b	Size-Groups: Minus No. 100	No. 100 to No. 50	No. 50 to No. 30	No. 30 to No. 16	No. 16 to No. 8	No. 8 to No. 4	No. 4 to ⅜ in.	⅜ in. to ¾ in.	¾ in. to 1½ in.
				Relative Densities								
				Criteria: Maximum Strength and Workability								
6	A	4.65	.432	.210	.112	.130	.172	.168	.127	.000	*.325*	
	8	5.70 - 4.30	.325	?			.172				.282	
		0.95					100%				115%	
7	A	4.83	.392	.190	.101	.120	*.174*	.159	.121	.000	.356	
	6	5.25 : 4.75	.356				.172				.356	
		1.20					101%				.100%	
8	A	4.87	.382	.178	.096	.115	.167	.145	.118	.000	.356	
	8	5.17 : 4.83	.356	?			.172				.356	
		1.25					97%				100%	
9	A	4.97	.343	.144	.081	.096	.144	.137	.107	.000	.353	
	8	4.93 : 5.07	.353	?			.172				.356	
		1.75					84%				99%	
10	B	4.91	.389	.016	.108	.314	*.194*	.130	.067	.000	*.359*	
	5	5.07 : 4.93	.359				.192				.356	
		1.20					101%				101%	
11	F	4.94	.355	.154	.085	.103	.150	.143	.110	.000	.354	
	6	5.00 : 5.00	.354				.151				.356	
		1.20					99%				99%	

Table continued page 28-11—opposite

Table C—continued

No.	Sand Fig.	F. M. Ratio Bbl.	a b	Size-Groups: Minus No. 100	No. 100 to No. 50	No. 50 to No. 30	No. 30 to No. 16	No. 16 to No. 8	No. 8 to No. 4	No. 4 to ⅜ in.	⅜ in. to ¾ in.	¾ in. to 1½ in.
				Relative Densities								
				Criteria: Maximum Strength								
12	C	4.80	.410	.046	.134	*.235*	*.266*	.120	.068	.000	.338	
	5	5.34 : 4.66	.338			.192	.192				.356	
		1.20					139%				95%	
13	D	4.42	.423	.106	*.203*	*.283*	.165	.074	.053	.000	.325	
	5	5.50 : 4.50	.325		.192	.192					.412	
		1.20				147%					79%	
14	E	4.43	.404	.073	.246	*.274*	*.215*	.031	.000	.000	.344	
	5	5.26 : 4.74	.344			.192	.192				.412	
		1.20				143%					83%	
				Criteria: Workability (Estimated)								
15	C	5.46	.285	.034	.097	.186	*.219*	.101	.058	.000	*.463*	
		3.73 : 6.27	.463				.192				.412	
		1.20					114%				113%	
16	D	5.43	.273	.080	.144	*.219*	.133	.060	.042	.000	*.475*	
		3.52 : 6.58	.475			.192					.424	
		1.20				114%					112%	
17	E	5.31	.268	.049	.182	*.219*	.180	.025	.000	.000	*.480*	
		3.45 : 6.55	.480			.192					.424	
		1.20				114%					113%	

oversanding with Sands A, B, and F increases the fines faster than the increase in particle interference and without disturbing the suspensive structure; the compressive strengths of their concretes remain horizontal to the left of the point of optimum mixtures.

TABLE D—CRITICAL SPACING OF SIZE-GROUPS

By formula for clear distance, t, between particles

$$t = \left[\left(\frac{d_o}{d_a}\right)^{1/3} - 1\right] D$$

Shape of Agg.	Dry-Rodded Voids			d_a, Values of Relative Density				
				Kind of Grading				
	Solid Vol. d_o	Voids in Unit Vol.	Voids per cent	Continuous	Groups in Gap			
					1	2	3	4
Diameter of Dominant Group				D	D	D	D	D
Next Smaller Diameter				½D	¼D	⅛D	1/16D	1/32D
Nearly Round	0.650	0.350	35.0	0.192	0.333	0.457	0.541	0.594
Sub-angular	0.615	0.385	38.5	0.182	0.312	0.432	0.515	0.565
Broken, Cubical	0.580	0.420	42.0	0.172	0.297	0.407	0.484	0.530
Broken, Disklike	0.550	0.450	45.0	0.163	0.282	0.387	0.458	0.503
Crushed Splinters	0.510	0.490	49.0	0.151	0.261	0.357	0.425	0.467
Corrected values because of poor suspension with gap gradings and settling of coarse particles in medium concrete used in tests under discussion for the broken cubical stones								
Values above					0.297	0.407	0.484	0.530
Reduction					5%	12½%	15%	20%
Corrected values					0.282	0.356	0.412	0.424

For wetter consistencies the values for gap gradings should be reduced from 5 per cent to 10 per cent more than with medium consistencies.

TABLE E—SIEVE ANALYSES

Interpolated from Fig. 2

Sands	F. M.	Minus No. 100	100–50	50–30	30–16	16–8	8–4
		Percentages in Size-Groups					
A, F	2.88	15	9	12	21	23	20
B	2.88	1	8	34	26	20	11
C	2.88	3	10	23	35	18	11
D	2.34	8	17	33	23	11	8
E	2.12	5	22	34	34	5	0

Coarse Agg.	F. M.	No. 4–⅜ in.	⅜–¾ in.	¾–1½ in.
¾ in.	7.00	0	100	0
1½ in.	7.50	0	50	50

TABLE F—METHOD OF COMPUTING RELATIVE DENSITIES

Data: Sand A with ⅜ to ¾ in. reshaped stone.
Bbl. per cu. yd., 1.20, gives $c = 0.085$.
W/C, by wt., = 0.62, or $w = 0.167$
Unit paste: $c + w = 0.252$, hence unit absolute volume of aggregate is, $a + b = 1.000 - 0.252 = 0.748$.
F. M. is 4.83, and the ratios of aggregates is 5.25 to 4.75, and $a = 0.392$, $b = 0.356$.

	Size-Groups							
	Minus No. 100	No. 100 to No. 50	No. 50 to No. 30	No. 30 to No. 16	No. 16 to No. 8	No. 8 to No. 4	No. 4 to ⅜ in.	⅜ in. to ¾ in.
Analyses of aggregates	15%	9%	$a =$ 12%	0.392 21%	23%	20%	$b =$	0.356 100%
Absolute volumes	.059	.035	.047	.083	.090	.078	.000	.356
Relative spaces	.311	.346	.383	.476	.566	.644	.644	1.000
Relative densities	.190	.101	.120	*.174*	.159	.121	.000	.356
Table DD limit				.172				.356

Absolute volumes are distributed to size-groups according to sieve analyses.

Each relative space is obtained by beginning at the right, subtracting 0.356 from unity and setting this remainder to the left and thereafter subtracting the abs. vol. from the relative space under it to give the space available to the particles at its left.

Each relative density is the ratio of the absolute volume to the relative space in the same column. Those italicized are dominant.

Underscoring represents a jump grading with two groups in the gap. A jump grading occurs for the group when the sum of the material in the group, or groups, to its left is less than from 22% to 26% of its absolute volume. Thus, in this example, 0.078 + 0.000, or 0.078, multiplied by 4 gives 0.312, which is less than 0.356.

It should be noted that the cohesive quality of these mixes is put to a severe test by the manner in which the compressive test cylinders were made. They were compacted by vertical jigging. Each drop of the vibrating table tended to settle the solids to an arched condition after which stratification of the cement could proceed. It is this stratification, as T. C. Powers has shown, which causes loss of strength for a given water cement ratio. The placeability tests, on the other hand, were made by horizontal vibration and more nearly agree with compaction in field practice.

It seems that the variation of strengths with consistency, noted by the author and shown in Fig. 3 and 4, can be attributed in part to this type of cylinder fabrication and in part to the greater facility for settling of solids in a wetter mix.

CLOSURE BY G. W. HUTCHINSON

Mr. Foresman's interpretation of the water-cement ratio law, that "any change in any of the ingredients other than the water and cement will set up a new W/C-strength relationship" is to restrict the application of the

law much more than is commonly done in practice. For example, a change in gradation of the aggregate should not upset the relationship, provided the mixture remains workable. Therefore the interchange between cement and −100 mesh dolomite, indicated in Table I of the paper should not require a different W/C-strength relationship, according to common interpretation. However, the data of the Table as plotted in Fig. 4 indicate that consistency has an effect apart from W/C.

The −100 dolomite consisted of aggregate regardless of the fact that it contributed to what would ordinarily be called the "paste" portion of the mix. It was coarser than the cement. It would be a part of the "aggregate range" referred to by Mr. Foresman. As regards discussion of gradation of total mix—the cement would also have to be included.

The nature of these investigations suggest classification as to "fine" and "coarse" aggregates to make the data more easily understood. The term "undersanded" immediately pictures the relative general condition to which reference is made.

Mr. Foresman states that "The strength of the concrete will follow the absolute volume of the aggregates up to a point where the volume of the aggregate voids equals the volume of the cement paste. *Beyond that point, the strength . . . will remain constant as long as the Water Cement Ratio is constant.*" This statement may seem to find confirmation in Fig. 7 of the paper, but the point which the data there plotted were designed to emphasize is that there is an optimum degree of placeability for a definite gradation, and departure either way from this optimum requires an increase in W/C (or a decrease in aggregate volume) to maintain equal placeability.

The author is indebted to Mr. Weymouth for his interest and analysis of the data presented in the paper. He predicates his analysis upon the fact that particle interference is the cause of decreasing the placeability of concrete where the author infers its probability at the top of page 22 in the paper. As stated by Mr. Weymouth, correlation of the data with respect to any theory governing aggregate gradation would be precluded. In outlining the investigations, the possibility of such correlation was not considered.

Stratification of material would be a possibility in placing concrete either in cylinders or in the field and by any means. It is felt that this action was not exaggerated when casting the specimens represented by Fig. 3 and 4.

With regard to the stone sand (A) used on the project, it was economical to use the natural gradation in the sizes above the No. 100 sieve with only such changes as would not cause waste, and to then improve placeability by the judicious use of the material passing the No. 100 sieve in

lieu of wasting this and increasing the cement content. Various other tests, including some in the former paper, "Concrete Aggregate Development on the Claytor Hydro Project," indicated that placeability, compressive strength, and durability could be greatly improved by the use of this finer aggregate in optimum amounts.

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A Concrete Failure Attributed to Aggregate of Low Thermal Coefficient*

By J. C. PEARSON†
Member American Concrete Institute

SYNOPSIS

This paper gives the high-lights of an investigation to account for a rapid failure of some cast stone steps in the winter of 1937-38. Preliminary studies of the cement, aggregate and concrete gave no clue to the trouble except to show that the aggregate in some way caused rapid failure of the concrete in freezing and thawing tests. Eventually the aggregate was found to have an unusually low thermal coefficient, although very resistant to frost action in itself. A number of other low coefficient aggregates were found, and in concrete subjected to freezing and thawing tests, all produced the same type of rapid failure. Such aggregates evidently produced high internal stresses at low temperatures and made the concrete particularly vulnerable to frost action.

Committees and individuals attempting to list the factors which affect the durability of concrete have frequently mentioned thermal coefficient of aggregates as one of those to be considered. So far as the writer is aware, no definitely established cases of failure from this source have been reported, although certain aggregates have been under suspicion from the relatively low resistance of concrete made with them in freezing and thawing tests. However, the following account of a cast stone failure contains rather convincing evidence that the low thermal coefficient of a dolomitic marble was primarily responsible for the trouble in this case, which was made spectacular by the contributing effects of an unusually severe exposure.

In the summer of 1937, some very imposing flights of cast stone steps were installed for the approaches to a University building situated on a hill top in a northern city. These steps were cast in about 8 ft. lengths, approximately 6 x 12 in. cross-section. The riser and tread faces were

*Received by the Institute Apr. 23, 1941.
†Director of Research, Lehigh Portland Cement Co., Allentown, Pa.

dressed to expose the aggregate, which was a crystalline dolomitic marble. The cement used was a mixture of 75 per cent H.E.S. and 25 per cent white portland, both of normal good quality so far as known. The cast stone was of exceptionally good quality as manufactured—the mixture was 1:4 by weight, with well graded aggregate, W/C of 5½ gal. per sack, and vibrated into rigid molds.

In April 1938, these steps were found to be shot with fine cracks on the exposed faces, crumbling had already started at the grade line, and the concrete was incredibly and swiftly on its way out. The appearance of the steps at that time suggested internal swelling as though from unsound aggregate, but the latter had been above suspicion, as it came from a source from which the manufacturer had obtained his supply for many years. Nevertheless, the aggregate was suspected, and some 200 typical pieces of the ¾ to ⅝ in. maximum sizes were immersed in water and subjected to freezing and thawing tests. After more than 300 cycles only five pieces of the aggregate had split, and there was practically no disintegration. One could not reasonably demand a more durable aggregate.

It will serve no useful purpose to go into the details of the extensive test program undertaken to account for this rapid failure. Samples of of the cement actually used in the concrete were not available, but the same brands from the same sources were used in reproducing the job concrete in the laboratory. These cements were tested with other aggregates, and other cements were tested with the dolomitic marble. Freezing and thawing tests proved conclusively that the marble aggregate caused a rapid failure of the concrete in which it was used, regardless of the type of cement.

Fortunately, a pair of 1½ x 1½ x 10 in. bars of the marble itself had been obtained, which were submitted to cycles of wetting and drying, heating and cooling, freezing and thawing. Nothing happened from a long course of such treatment, except that as a result of heating and cooling, the marble showed a slight growth of one or two hundredths of one per cent. In some way this suggested making a determination of thermal coefficient between the temperatures 0 and 120 degrees F., within which range all the previous tests had been conducted. Unexpectedly low values were obtained, and later determinations with more refinement showed that the thermal coefficient was very close to .000002 per degree F.

This was believed to be the key to the failure. The thermal coefficient of cement paste is variable according to Meyers(*), depending upon the quantity of gel present, its state of desiccation, age, etc.; the Bureau

(*) S. L. Meyers: Thermal Coefficient of Expansion of Portland Cement. *Industrial & Engineering Chemistry*, Vol. 32, p. 1107, August 1940.

of Reclamation(*) gives values varying from .000006 to .000009 per degree F. To make a very rough estimate of internal stresses, let us assume that the coefficient for the cement matrix in the cast stone was .000007. Between paste and aggregate, then, there existed a linear differential deformation of some five millionths for each degree change in temperature. A possible drop of 100° would give a relative deformation of .0005 in. per inch, and for an assumed modulus of 4,000,000 p.s.i. for the paste, the tensile stress in the latter would be 2000 p.s.i. without allowance for aggregate deformation and for creep. The stress so computed is probably too high, and in fact might be more logically taken at about ⅔ of this value, as the modulus of the aggregate was determined as about 8,000,000 p.s.i. On the other hand, as will be indicated later, it is possible that the thermal coefficient of the aggregate was considerably lower at low temperatures than the value of two millionths given above, so that it is impossible to make an accurate estimate of the tensile stress in the paste. It seems very probable, however, that the stresses were at times sufficient to produce cracks in the matrix. This would have made the concrete particularly vulnerable at low temperatures and afforded opportunity for effective frost action.

To secure further evidence in support of the foregoing deduction, the thermal coefficients of quite a number of aggregates that were suspected of having low coefficients were determined, and several having values as low as that of the dolomitic marble were found. In this search it became apparent that marbles and crystalline limestones very frequently have coefficients that diminish with temperature, and that such coefficients may actually become negative within the range of temperatures common to the northern part of the country. Therefore, it seems important, in relation to the question of concrete durability, that the thermal coefficients be determined particularly for temperature ranges from room temperature to — 20 F. or lower.

Table 1 gives the coefficients of four aggregates, the first three of which have been used in the commercial production of cast stone. For these three aggregates, coefficients were determined for ranges both above and below room temperature, and it is immediately apparent that one could be easily misled in judging the acceptability of the Maryland marble, for example, from the coefficient as determined for the range 75 to 212 F.

These four materials were crushed and screened to give a graded coarse aggregate between ½-in. and No. 4. They were combined with a mortar consisting of normal cement and fine Ottawa sand and cast in 2 x 2 x 10-in. bars with steel contact points embedded in the ends.

(*) Bureau of Reclamation. *Concrete Manual*, p. 37.

TABLE 1

Thermal Coefficients per Degree F. ($x10^6$)

Material	Temp. Range (F.)	Coefficient
Marble (N. Y.)	—25 to 75°	2.5
Marble (N. Y.	75 to 212°	4.2
Calcite (N. J.)	—25 to 75°	0.0
Calcite (N. J.)	75 to 212°	3.7
Marble (Md.)	—25 to 75°	—1.2
Marble (Md.)	75 to 212°	4.4
Vycor Glass	—20 to 180°	0.5

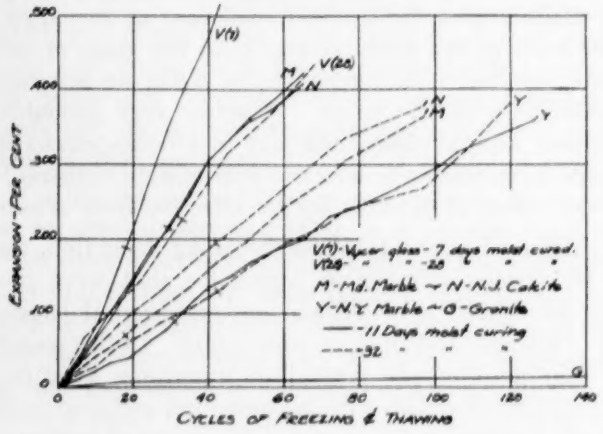

Fig. 1

These bars were cured for two different periods, and then submitted to freezing and thawing tests. The lengths were measured every few cycles and their behavior is shown by the plot of expansion versus cycles in Fig. 1.

In general these tests were continued as long as the bars could be handled without breaking them. The little cross lines on the lower portion of the curves indicate the points where one or both of the steel contact points became loose. When it is realized that an expansion of .1 to .2 per cent represents practical failure, it is apparent that these aggregates cause very rapid failure under the conditions of the test. For comparison, the curve marked G shows normal behavior of good concrete in such tests, although the G bars were not a part of this particular series.

It is perhaps worthy of note that while the life of the bars tends to be shorter, the lower the aggregate coefficient, the surface condition of the aggregate also plays a part. Thus the Vycor glass has smooth

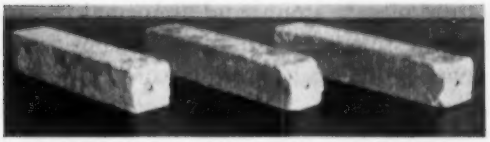

Fig. 2—New York marble, New Jersey calcite and Maryland marble bars (left to right) after 35 cycles of freezing and thawing.

glassy surfaces, and correspondingly low bond, and gave the mostr apid failure. The New Jersey calcite is also characterized by smooth crystal faces, and although its low-range coefficient was above that of the Maryland marble, its performance was about the same. The New York marble had both a more favorable surface condition and a higher coefficient, and succumbed to the frost action somewhat more slowly.

Fig. 2 shows three bars after 35 cycles of freezing and thawing, containing respectively the New York marble, the New Jersey calcite, and the Maryland marble (from left to right). The expansions ranged from .1 to .25 per cent and the bars were therefore in bad condition. The photograph shows, however, that there was very little loss of weight, which suggests that loss of weight as a criterion of failure in failures of this type would be inadequate to represent the true condition of the specimens.

It may, therefore, be concluded that the low coefficient of the dolomitic marble in the case described was the prime cause of the failure of the cast stone steps. Nevertheless, there was another factor which no doubt contributed materially to the rapid disintegration, and that was a structural detail. These steps were set in mortar on a monolithic reinforced concrete foundation having the same profile as the finished steps. It is obvious that rain and melting ice and snow would tend to keep this mortar bed, and to a somewhat less degree, the steps themselves, more or less saturated for long periods of time between fall and spring. In the severely exposed location, the conditions to which these steps were subjected were quite analogous to those obtaining in the laboratory tests, and the relatively small number of freezing and thawing cycles required to damage this particular kind of concrete could readily have occurred in a single winter. Another factor was also favorable to the vulnerability of these steps, in that the cast stone was an unusually dense concrete, cast in mid-summer. Therefore, it is to be assumed that internal stress developed and increased more or less steadily as the winter approached, the really damaging stresses occurring at freezing temperatures or below, and causing the concrete to be most susceptible to frost action just when frost action could occur.

While there is no need to take an alarmist view of the possibilities of damage from low coefficient aggregates, this matter should be of particular concern to the cast stone industry. Marble aggregates are

commonly used in the manufacture of cast stone, and abnormal thermal behavior in marbles seems to be the rule rather than the exception. If such aggregates are used in severely exposed exterior work, the possibility of trouble should not be overlooked. The information gathered in the course of this study suggests a probable cause for a number of hitherto unexplained cast stone failures.

In the general concrete field there is less likelihood of trouble from aggregates of abnormal thermal coefficients. Examination of existing data suggests that very rarely are coefficients *high* enough to cause any worry, but the data at low temperatures are too few to indicate with certainty whether some aggregates may not have undesirably low coefficients. It seems fairly certain that the danger lies in aggregates of this type, which cause high internal stresses at *low* temperatures, and therefore are especially to be avoided in concrete that is frequently or continuously wet and at the same time subject to severe frost action.

Discussion of this paper should reach the ACI Secretary in triplicate by Mar. 1, 1942, for publication in the JOURNAL for June, 1942

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Discussion of a paper by J. C. Pearson:

A Concrete Failure Attributed to Aggregate of Low Thermal Coefficient*

CONVENTION DISCUSSION: J. C. PEARSON, F. B. HORNIBROOK, H. S. MEISSNER, R. B. YOUNG, B. KELLAM, R. E. DAVIS, M. O. WITHEY, M. A. SWAYZE, C. G. WALKER; SINCE CONVENTION: F. V. REAGEL and T. F. WILLIS—AUTHOR'S CLOSURE

By MR. PEARSON,
(Introducing Convention Discussion)

As the substance of this paper was presented at the meeting of Committee 115, Research, a year ago, and the paper itself published in the September 1941 JOURNAL only a very brief review will be presented here with a few additional remarks on later developments.

The case had to do with an installation of exceptionally well made cast stone steps as the approaches to a University building in the northern part of the country. The steps were cast and placed in the summer of 1937; in the spring of 1938 they were found to be more or less covered with fine cracks and in a well advanced stage of disintegration. The appearance of the concrete suggested the effects of an internal swelling, but there was no suspicion of faulty material in the ingredients. The location and the details of construction were such as to contribute to the severity of the exposure.

Tests were undertaken immediately to reproduce the job concrete in the laboratory. A rather coarsely crystalline dolomitic marble was used as aggregate, and it was found that all test specimens containing this aggregate failed very rapidly from alternate freezings and thawings. It was thought that the aggregate itself might have low resistance to freezing and thawing but this was found not to be the case; in fact the aggregate was not appreciably affected by 10 times the number of cycles which destroyed the concrete.

The only clue to the trouble eventually came from length measurements of some bars cut from the marble, while exposing them alternately

*ACI JOURNAL, Sept. 1941; *Proc.* V. 38, p. 29.

in a refrigerator at zero F. and in an oven at 120 F. The coefficient of thermal expansion over this temperature range was found to be .000002 per degree F., and it was therefore obvious that rather high internal stress in the concrete should be expected to develop in cold weather. During the fall and winter it is to be presumed that the thermal contraction of the matrix, linearly, was 3 to 4 times that of the aggregate. Except for the mitigating effects of creep, which cannot even be estimated in this case, computations show that the tension set up in the cement matrix was probably sufficient to produce cracks therein at winter temperatures. While these cracks must have been originally very fine, perhaps even submicroscopic, they would permit the entrance of water or melting snow and ice at the very time that freezing could do most damage.

The obvious thing to do after this theory was established, was to repeat the work more intensively with this aggregate, but when additional samples were obtained in 1939, they were found to have higher coefficients and were therefore unsuited for the further investigation. Progress being blocked in this direction, a search for other aggregates of low coefficients was made. Eventually four others were found and these were used in the place of the dolomitic marble. The details of these studies are given in the paper; it is sufficient to state here that the results seemed to confirm the theory in all respects, indicating that concretes containing aggregates with coefficients lower than about .0000025 per degree F. were particularly vulnerable to frost action under wet conditions.

The startlingly rapid failure of the job concrete indicated that setting the steps on a reinforced concrete foundation in such manner that water could not readily drain away from the bed mortar must have been a potent factor contributing to the speed of failure. There is indirect evidence of this from a set of 2 x 2 x 10-in. bars made with the original marble aggregate and exposed to rain, sun, wind, snow and ice on the slightly pitched roof of a storage building adjoining our laboratory in Allentown. These bars were thus exposed for more than three years, and up to December, 1941, they had expanded only from 0.04 to 0.09 per cent. At this time they showed a few fine cracks, which from casual observation might have been taken for ordinary crazing, and did not appear to be indicative of severe internal disorder. Yet the expansion was definite evidence that they were failing gradually, and in fact the bar with the highest expansion might be considered close to failure. One of these bars, which early in December, 1941, had expanded 0.06 per cent, was transferred to a tray of water on the roof, in which the average depth was about 1½-in. During the succeeding two months there were recorded 18 cycles of temperature through the freezing point. Not all of these could probably be considered effective freezing and thawing cycles,

yet during that period the bar expanded from 0.06 to 0.23 per cent, after which it was easily broken in the hands. These observations thus seem to indicate that severity of exposure is an important factor in the durability of concrete containing aggregate of low thermal coefficient. Without frequent saturation of such concrete under freezing conditions, deterioration would be gradual, and might not even be suspected until failure was imminent.

By F. B. HORNIBROOK*

In recent years a disquieting number of instances of concrete disintegration has been pointed out which may be attributable in whole or in part to the aggregates. In some cases the disintegration was due to an unfortunate combination of certain aggregates with certain cements where incompatible chemical properties were brought together, as in the aggregate-alkali reaction; in other cases the disintegration was caused by the specific inability of the aggregate to withstand frost action, as in some cherts. Mr. Pearson has now commendably pointed out another cause of disintegration, namely, that which results when, under certain conditions of exposure, the thermal expansivity of the aggregate is incompatible with that of the associated materials.

An example of disintegration wherein the low coefficient of thermal expansion of the aggregate may have played a part was also found in a high school stadium built in a mid-western Pennsylvania town during the summer of 1940. One year later much of the exposed area of the

*National Bureau of Standards, Washington, D. C.

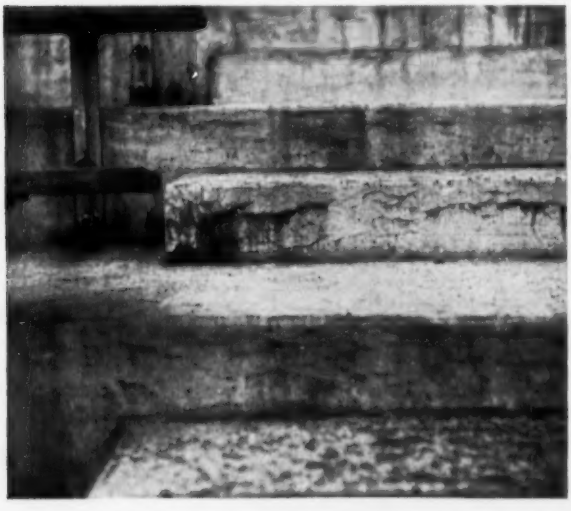


Fig. A—Typical appearance of stadium after one winter's exposure.

stadium exhibited extensive disintegration of a type illustrated by Fig. A. In addition to the disintegration shown, surface cracks were widespread on the tread and fine cracks were abundant even on the sides and top of the parapets. The concrete was easily broken away with a hammer at the corners of the steps, separations occurring quite clean between the coarse aggregate and the matrix.

The mix used was 1:2:3½ by volume with 5½ to 6 gal. of water per bag of cement. A crushed fine-grained crystalline limestone was used for both the fine and the coarse aggregate.

The factor of workmanship was, of course, indeterminate. The same contractor, however, had built a municipal stadium 2 years before, which was in nearly perfect condition. There was no tendency on the part of the local supervisor nor of a school board member that watched the work closely, to blame the contractor.

The concrete probably was fairly harsh for placing; the water content had to be kept down since forms were provided for risers only and the concrete had to be kept stiff enough to keep it from flowing from a higher step to a lower one. The crushed stone fine aggregate had only 5 to 6 per cent (5.3 per cent for the sample tested) passing the No. 100 sieve. It seems probable, therefore, that the surfaces, and particularly the edges, had to be worked considerably, thereby resulting in an excess of mortar at the surface. This was found to be true for several samples taken from step edges.

Fortunately, samples of the original cement and also of the fine and the coarse aggregates were available. A section of one of the steps was also obtained. Nothing unusual was found in the chemical analysis of the cement and aggregate, from autoclave tests of the cement, or from petrographic examination of the aggregate and hardened concrete. Tests for reaction between the aggregate and the 0.67 per cent total alkali in the cement were entirely negative. Although the concrete in the step taken for tests was so weak near the top that cores could not be successfully cut, two cores cut from the bottom had compressive strengths of 5400 and 6070 psi. A section 6 x 2½ x 1¾ in. cut from the bottom had a flexural strength of 700 psi.

Freezing and thawing tests on specimens made from the original cement and aggregate indicated resistance that averaged as good or better than so-called normal concrete similarly fabricated and cured.

Measurements of the thermal expansion of the coarse aggregate were made by W. H. Johnston of the National Bureau of Standards over the range − 4 to + 140 F. Between − 4 and 32 F the average coefficient of thermal expansion of two specimens was found to be 2.0×10^{-6}; the aver-

age coefficient between 32 and 140 F. was 2.4 x 10^{-6}. As pointed out by Mr. Pearson, and also by R. E. Davis* and by Willis and De Reus†, this is considerably lower than the coefficient for cementitious material.

Meyers‡ found marked differences in the coefficient of expansion with richness for a limestone concrete; thus a 1:7.5 six months old limestone concrete had a coefficient of 3.9 x 10^{-6} whereas a richer 1:3.1 concrete had a coefficient of 7.8 x 10^{-6}. Such a difference in coefficient with richness could be responsible for excessive stress between an overworked surface and the leaner interior.

The stadium concrete was poured starting in June, when of course the temperature was fairly warm. In the fall large daily temperature variations occur in that section of the state. No weather reports are available from the town in which the stadium is located but official records from a nearby town show a maximum daily variation of 46 and a mean daily variation of 28.8 F. for the month of October. The exposure of the stadium is to the southeast, hence the daily variation in the temperature of the surface of the concrete was undoubtedly much greater than the official temperature variation, particularly on clear days. The weather reports for the nearby town also indicate 14 possible cycles of freezing and thawing for October and 18 cycles for November. Weather records for other months were not obtained.

The mechanism of disintegration therefore may have been somewhat as follows: Large temperature differences probably developed at times between the surface and interior, and the difference in coefficient of expansion between the mortar-rich surface and the interior may have caused fairly large tensile stresses at the surface; sufficient, possibly, to open the fine cracks that were so evident over most of the exposed structure. The steps were probably near saturation a good part of the late fall, and these fine cracks could have provided an opening for water. The freezing and thawing action then probably caused the actual softening and disintegration.

An interesting application of marble as aggregate is to be found in a commercial cast stone coping and belt course placed in 1940 on an extension to one of the Buildings at the National Bureau of Standards. The marble, a coarse-grained crystalline friable material from near Baltimore, Md., was used in the proportions 1 part cement, 1 part river sand to 2 parts marble sand graded up to $\frac{3}{8}$ in. Thermal expansion measurements by W. H. Johnston on crystals of the marble selected by W. H. Parsons for their orientation along the principal crystallographic axes,

*A Summary of Investigations of Volume Changes in Cements, Mortars and Concretes Produced by Causes other than Stress, Proc. A.S.T.M., 30, I, 668 (1930).

†Thermal Volume Change and Elasticity of Aggregates and Their Effect on Concrete, Proc. A.S.T.M., Vol. 39, p. 919, 1939.

‡Ibid, p. 1109.

indicated a coefficient of thermal expansion very similar to that of pure calcite; that is, a high positive coefficient along one axis and a negative coefficient along the other axes. From − 4 to 140 F. the coefficient of expansion of the selected crystals, parallel to the C-axis, was 12.5×10^{-6} and perpendicular to the C-axis was -2.2×10^{-6}. The calculated coefficient of volume expansion would therefore be 2.7×10^{-6}.

Several additions, water contents, and curing conditions were used during the fabrication of the cast stone and a complete record was maintained. From time to time various measurements are being made on this cast stone in place so that eventually an excellent case history should be available to help in diagnosing the cause of trouble should any develop. The only unsatisfactory behavior observed thus far is a pronounced crazing on many of the stones, the cause of which has not yet been established.

The work of Mr. Pearson, as well as that of others, on the behavior of aggregates in concrete, emphasizes the need of considering the mineralogical properties of the material to be used as aggregate in relation to the properties of associated materials and in relation to the use and exposure of the concrete. In many cases this will require a detailed classification of the minerals making up the rocks in the aggregate, as well as a study of the rock textures.

By H. S. MEISSNER*

Mr. Pearson's paper recalls some freezing and thawing tests which had been conducted in the Bureau of Reclamation laboratories some time ago. A comprehensive series comprising several aggregates, using one cement at the same water-cement ratio, varied widely in relative durability. No reasonable explanation for all the differences in resistance to freezing and thawing could be discerned from the absorption, air, water content, cement content, or other characteristics of these concretes. There was likewise no correlation between these tests and the relative soundness of the aggregates as determined from sodium sulfate and freezing and thawing tests; in fact some of the more resistant concretes in the group were composed of the least sound aggregate as reflected by the sodium sulfate tests. The only conclusion that could be drawn from this work was that differences in the concretes were due to inherent differences in the aggregates.

A study of the petrographic composition of these aggregates did, however, at first suggest a speculative answer. It was observed that those concretes whose aggregates consisted entirely of one rock type, or preponderately so, were amongst the most durable, and that, on the other

*Engineer, U. S. Bureau of Reclamation, Denver, Colo.

hand, the concretes containing a heterogeneous aggregate, composed of a miscellany of minerals and rock types, were the least durable. Such grouping suggested that differential expansion and contraction within those concretes composed of assorted aggregate particles, each with different coefficients of expansion, contributed to their failure.

Accordingly, an attempt was made to check such theory by fabricating several groups of 3 x 6-in. concrete test specimens, in which all members of each group were made with a uniform aggregate prepared by crushing a single type of rock. An additional group was then made, composed of a mixed aggregate including equal parts of all the others. Crushed limestone, granite, quartzite, basalt, and sandstone were the rock types used in this series. All were graded alike and the same cement at a fixed water-cement ratio was used for all groups. The paste content was adjusted to give the same consistency in all groups, but this variation was not great, and it was felt that the concretes were enough alike in all respects other than the composition of the aggregate to permit the study of this variable. In the test, the specimens were placed in water, within rubber tubes, and frozen in cold circulating brine at a rate such that their centers reached 15 F. in one hour and 20 minutes from an initial uniform temperature of 70 F. Thawing to 70 F. was accomplished in the same elapsed time by placing the tubes in running tap water.

When subjected to freezing and thawing, these groups showed wide differences in durability, the limestone having twice the resistance of the basalt concrete and the others ranging between these two. However, contrary to the assumption first made, under which the concrete containing the mixed aggregate might be expected to have the least durability, this concrete was found to give results practically equal to the mean of all the other groups. And yet, if the extremes of durability may be attributed to extremes in thermal coefficient of the aggregate, an average durability might be expected from a combined aggregate in which those extremes are compensating. However, the thermal coefficients of the basalt (.0000032) and limestone (.0000043) were not found to differ greatly, although they are in the order suggested by Mr. Pearson. Thermal coefficients for the other rock types were not determined.

Another set of data, wherein several aggregates were used in identical concretes submitted to freezing and thawing tests, have been examined to see if they would conform to Mr. Pearson's proposition. These were crushed aggregates, graded alike, from minus 100 mesh to ¾-in. maximum, and used with the same cement at the same water-cement ratio to make 3x6-in. concrete test cylinders. The thermal coefficients for these aggregates had been determined and are listed in Table A, along

with the number of freezing and thawing cycles which the several concretes had endured before reaching the degree of failure noted.

TABLE A

Type of Aggregate in Concrete	Thermal Coefficient in. per in. per Degree F.	Number of Freezing and Thawing Cycles	
		To 25 Per Cent Loss in Weight	To 0.3 Per Cent Expansion
Diorite No. 1	0.0000047	13	6
Quartz diorite	0.0000044	25	19
Limestone A	0.0000023	44	41
Diorite No. 2	0.0000038	59	45
Limestone B	0.0000018	84	88

The concretes are arranged in the order of durability, and it may be noted that the concrete containing the aggregate of lowest coefficient was the most resistant, and that the aggregate in the least durable concrete had the highest thermal coefficient. With one exception the other concretes were in this same order. This series of tests, therefore, fails to support the theory advanced in the paper.

Supplementing the groups shown above, and identical in all respects except for the cement used and amount of curing prior to freezing and thawing, is another series (Table B) wherein resistance may be compared to the coefficient determined for the aggregate. In this series repeat tests were made with another cement. Limestone C was found to have a thermal coefficient of 0.0000034 inch per inch per degree F. parallel to the bedding plane in the rock, and 0.0000053 perpendicular to the bedding plane.

TABLE B

Cement	Type of Aggregate in Concrete	Thermal Coefficient	Number of Freezing and Thawing Cycles	
			To 25 Per Cent Loss	To 0.3 Per Cent Expansion
A	Dolomite	0.0000045	535	540
	Graywacke	0.0000051	105	65
	Limestone C	0.0000044	18	10
B	Dolomite	0.0000045	351	351
	Graywacke	0.0000051	148	134
	Limestone C	0.0000044	40	50

It is observed that the concrete which was the least durable in the group had aggregate with the lowest coefficient. The range of thermal coefficient for these three aggregates does not appear to be large enough to account for the great difference in durability. Yet the relative resistance of these concretes must be attributed alone to the materials used in them. From this group it appears that the explanation for resistance, or lack of resistance, to freezing and thawing, which these materials impart to concrete, is to be found in some unrecognized quality or characteristic. It may be that surface texture, with some influence on bond between the aggregate and paste, is in great measure responsible for the degree to which concrete resists the destructive effects of freezing and of thermal alternations. To the writer this seems to be exhibited by tests on a crushed pegmatite quartz. Concrete specimens, similar to those mentioned, made from this aggregate fail completely within 40 cycles of freezing and thawing, although this material could hardly be touched when tested for soundness by sodium sulfate or freezing and thawing. Its thermal coefficient was determined to be 0.0000057 in. per inch per degree F., which I believe Mr. Pearson would not consider unsatisfactory. However these crushed particles have hard glassy surfaces, with which it may be imagined bond would be poor.

In spite of the above remarks, which may give the impression that the writer disagrees with the author, it is felt that the theory advanced has a good scientific basis. High internal stresses must occur when materials with widely different thermal expansions are yoked together and then subjected to alternations in temperature, and it is axiomatic that such a condition would induce strains contributing to failure. The object of this discussion is merely to present data bearing on the subject, whether the same gives support to the paper or not. Mr. Pearson deserves commendation for this interesting article and the ingenious explanation given for the particular case of concrete deterioration. It should serve as a stimulus to other investigators and might well be applied to many concretes which, although possessing everything that may be expected in good workmanship and design, fail to give reasonable service.

By R. B. YOUNG*

Mr. Pearson's paper points out a very much neglected property of natural rock; namely, its coefficient of expansion. It is very evident that if the coefficient of the aggregate be very greatly different than that of the cement paste, whether higher or lower, it would tend to set up destructive stresses in the concrete in the first case at the higher temperatures, in the second, at the lower. The severity of the stress set up

*Hydro-Electric Power Commission of Ontario, Toronto, Ont.

by the differences in the rates of volume changes would depend on the range of temperature experienced and the length of the exposure under substantial differentials of temperature. Thus, in warmer climates, the aggregate with the high coefficient of expansion would become dangerous; in the colder climates, the aggregate with the lower coefficient of expansion would be expected to give trouble.

The speaker had hoped to be able to describe another example of the destructive effects in concrete of a dolomitic limestone aggregate having a low coefficient of expansion. The failure of the concrete in question was undoubtedly caused by the aggregate. In preliminary tests of the suspected aggregate, certain specimens were found which acted as described by Mr. Pearson. But, subsequently, on another group of specimens prepared from new samples, no further examples of this type of rock were discovered. While these studies did not yield any conclusive results, they did result in developing a method of test different from that used by Mr. Pearson which we believe is worth recording.

In our earlier studies, we followed the technique of Mr. Pearson and used small prisms, but they were found to be difficult to caliper accurately as they were small in volume and susceptible to warping while handling. Later a cube specimen was substituted which eliminated the distortion and permitted measurements to be taken in three different directions, thus detecting differences in the coefficient of expansion with and across the grain of the rock. Barney Kellam of our staff, who worked out the technique of these tests, has prepared a brief description of the method used which will form a part of the discussion of Mr. Pearson's paper; so I will not attempt to describe it.

Using this method wherever we can get samples of suitable size, we are determining the coefficient of expansion of all samples of rock coming into our laboratory with a view to learning more about the destructive possibilities of aggregates with unusual thermal properties.

By B. KELLAM*

The measurement of the coefficient of expansion without elaborate equipment practically limits one to the use of an accurate machinist's micrometer and makes necessary the development of the ability to make rapid measurements with it at various temperatures. After some experimentation, specimens of cube shape were selected in preference to prisms which distorted with temperature changes. The method now in use in the H.E.P.C. Laboratory is as follows:

The stone is cut into 4-in. cubes which need not be accurately sawed or finished. Pins made of $\frac{1}{8}$-in. diameter cold rolled steel, rounded at one end and exactly $\frac{5}{8}$ in. long, are gently driven in holes drilled in the

*Hydro-Electric Power Commission of Ontario; discussion presented by Mr. Young.

faces, until they are embedded approximately $\frac{3}{8}$ in. leaving $\frac{1}{4}$ in. exposed. One pair of such pins serves to measure changes parallel to the strata on which the rock lies and one pair at right angles to the bed.

Dimensions are first read when the specimens have remained at room temperature for at least one day after being received. They are then placed in a refrigerator, each specimen with a thermometer attached to it by means of plasticene. The temperature cycle is 48 hours in the refrigerator, 24 hours at room temperature, 48 hours in a constant temperature oven at 215 F. and finally another 36 hours at room temperature. Readings are taken at the end of each of these different periods.

These measurements were made with a micrometer which reads to 0.1 mil. When a number of specimens are being handled, readings are greatly facilitated by having all cubes approximately the same in dimension. It was found necessary to insulate the frame of the micrometer from the hand as the heat from the latter would cause errors as much as 0.6 mil. after one minute of contact. When ready to make measurements the micrometer was set approximately to the known dimension, the specimen quickly taken from the oven or refrigerator and measured. If taken from the refrigerator, the thermometer attached to the specimen was quickly read.

Measurements were always made at room temperature between oven and refrigerator to check on any peculiarities that the stone might exhibit.

Two cycles were usually run, allowance being made for the expansion coefficient of the steel pins. Changes were calculated on the micrometer measurements minus the length of both pins after the latter had been corrected for temperature.

This method was found to be accurate and speedy, requiring no precision equipment to cut the stones since the latter need not be accurately cubed nor the faces be parallel; and the type of micrometer used is generally available in most testing laboratories.

By R. E. DAVIS*
(presiding officer)

The discussion has been very interesting. I have felt for a long time that we were neglecting some of these things such as the thermal expansion of the aggregate. I do not believe we can yet pin lack of durability entirely on the low coefficient of the aggregate. Other factors come in, such as surface texture and the mineral character of the aggregate itself. I believe one factor which has not been mentioned here is the rigidity, or modulus of elasticity of the aggregate; among our stones we find very large variations in modulus of elasticity, and naturally the

*Professor of Civil Engineering, University of California, Berkeley.

more rigid the material, other things being equal, the higher would be the concentration of the localized stresses set up as concrete goes through a cycle of high to low temperature. So far I think we have had in mind those localized stresses which are produced in a combination of hardened cement paste with a relatively high coefficient of thermal expansion and an aggregate with a relatively low coefficient of thermal expansion. On the other hand, in a structure restrained from free movement as, for example, a dam, the average structural stress due to a given thermal change would be less for an element composed of concrete containing low coefficient of expansion aggregate than for a corresponding element containing high coefficient of expansion aggregate, since the tendency of any concrete element toward overall expansion is largely determined by the aggregate. I wish to bring out this point because I have seen what seemed rather notable examples of structures in climates not subjected to freezing and thawing, that have been surprisingly free from cracks, where I know that the aggregate used had a very low coefficient of thermal expansion. Is there any further discussion on this subject?

By M. O. WITHEY*

I have not a scientific analysis to present, but simply this lesson from the structural engineer's standpoint: High ductility in structural steel has been required to provide a metal that will accommodate itself, without failing, to stresses beyond the limit of elasticity which are often developed during the construction. Ductility and toughness are also important qualities of mortar and concrete. In addition to investigating the expansion, surface texture and other characteristics of the aggregate which influence the ductility and toughness of concrete, I think we should strive to improve the ductility and toughness of pastes. If such improvements are made in pastes, we will be able to use safely aggregates with marked differences in coefficients of expansion.

By M. A. SWAYZE†

I became aware of Mr. Pearson's work on this subject soon after he discovered what was the trouble with his cast stone, and I want to thank him publicly for what he found. We too had a job of cast stone in central New York. Although it was beautiful looking stone when it went in, by the next spring it was in very bad condition. That cast stone was made up, as far as its coarse aggregate was concerned, of almost pure calcite. The calcite grams were practically all single crystals, from ¼ to ⅜-in. in size. After Mr. Pearson had called our attention to the effect of a low expansion coefficient, we looked up the thermal

*Professor of Mechanics, University of Wisconsin, Madison.
†Director of Research, Lone Star Cement Corp., Hudson, N. Y.

coefficients of calcite. Fortunately some work had been done on that. Mr. Hornibrook called it to your attention today, but perhaps the full significance of his remarks did not sink in.

Calcite, according to the text books, has a thermal coefficient of 14 millionths per degree F. along one crystal axis, and perpendicular to this axis a shrinkage of 3 millionths per degree rise in temperature. With cement paste having a coefficient of about 8 millionths, this means that some tremendous strains are set up when a calcite crystal is embedded in cement paste and put through a 100 degree rise in temperature. In one direction the crystal will expand 100 times the difference between 14 and 8 millionths, or 600 millionths of an inch per inch more than the paste. In the other direction it will shrink away from the paste to the amount of 1100 millionths. No cement paste can be expected to accommodate itself to such enormous differential changes.

From the appearance of the bad cast stone which we examined it was very apparent that this differential volume change was the cause of failure. Every calcite grain was torn completely loose from its moorings in the cement base, with no adhesion whatever of the paste on any of the crystal faces, and under the microscope the paste showed signs of disruptive strain.

We secured samples of a number of calcitic aggregates used in cast stone, and attempted to work out some sort of simple accelerated test which did not involve freezing and thawing. We were not very successful. We made our comparisons between these calcite aggregates and a gravel aggregate of equal size, using two ranges of heat change. One was our regular − 20 to + 70 F. cycle of freezing and thawing; the other was in the range from 70 to 170 F. In both cases the specimens were constantly immersed in water. While there was a considerably greater drop in strength of the concrete containing calcitic aggregates after 50 of the hot cycles, individual differences were not great enough to furnish criteria of good and bad. So far as we can see, the only way we can tell definitely if an aggregate is going to cause this trouble in cast stone is by an actual freezing and thawing test. While the differential volume changes between these aggregates and paste are sufficient to break the bond between them in any range of temperatures, the severe disintegration shows up only when water freezes in the cracks so formed. The performance record of a particular calcite aggregate in cast stone may be excellent for interior work or in vertical outdoor exposure, and yet it may be absolutely unfit for use when the stone is laid where it will become saturated with water and then freeze.

Regarding Mr. Meissner's remarks, I would like to know more about his limestones which did not seem to follow Mr. Pearson's theory. The

point here is that limestones are made up of individual calcite crystals in every possible orientation. If these crystals are very small, then the differential expansion between crystal and crystal will be so small that it will not break the bond between the particles of limestone nor between limestone and paste. It is only when individual crystals become large that this happens. Therefore I suspect that the limestones which Mr. Meissner found poorly durable were coarse grained, while those with good durability were fine grained.

This whole matter of differences in thermal expansion is one of extreme importance. It is regrettable that so little information exists on differential expansion along the axes of minerals. We have an outstanding difference reported for calcite. Dolomite and magnesite are closely allied to calcite, have wide differences in many physical properties from one mineral axis to another, and therefore should also show similar differences in thermal expansion. In general, any mineral which shows a large difference in indices of refraction from one axis to another must be under suspicion. If the crystal structure in aggregate particles bearing these minerals is large, then there must be grave doubt as to their ability to stand up under frost action when in a saturated condition.

MR. MEISSNER

I can assure Mr. Swayze that the limestone, about which he inquires, was a fine grained limestone. In this same connection, I would point out that the limestone C in Table B,* which is shown therein as having a coefficient of .0000044, was found to have a coefficient of .0000034 parallel to its bedding plane and .0000053 perpendicular thereto. Whether or not this difference within the stone itself is sufficient to account for its low durability, I am not prepared to say, but it furnishes interesting comment to Mr. Swayze's remarks.

By C. G. WALKER†

I happen to be familiar with those jobs of disintegrated cast stone that have been referred to here, and several others. I also happen to know that the aggregate used in those jobs has been used in many other jobs which have never given any trace of trouble. I would not question the theory. The expansion of aggregates no doubt has something to do with the durability of concrete, but I would like to have it borne in mind that there have been many instances of the uses of these particular aggregates which have not resulted in failure. I should also like to ask Mr. Pearson whether he believes that under the conditions which prevailed on this particular job—the use of an aggregate of low thermal

*See Mr. Meissner's discussion.
†Assistant Secretary, Cast Stone Institute, Washington, D.C.

coefficient and a very poor structural condition—which of the two contributed the more to the failure. If it was the structural conditions, then should we not put more emphasis on the correction of such practices rather than on theoretical consideration of properties of the aggregate which are in what we might call the fourth decimal place. I wonder if even a perfect aggregate would have made cast stone which would have stood up under the structural conditions on this job.

MR. PEARSON

Mr. Walker's question regarding the effect of adverse structural conditions has been answered in the paper, at least to the extent that it can be answered. If it had been possible in some way to prevent the accumulation of water behind and under the cast stone steps, I do not think they would have failed so rapidly. Therefore, in this case there was undoubtedly a considerable contribution from the environment, but even so, any concrete as well made as these steps were made ought to last considerably longer than one winter. The fact that some of the bars made to reproduce the job concrete failed rapidly in freezing and thawing tests, while others of the same lot were fully exposed to the weather on a slightly pitched roof for three years without showing much visible evidence of distress, indicates that the job exposure was extremely severe.

But I do not think Mr. Walker or anyone else should foster the notion that this low thermal coefficient effect is simply of academic interest—those very bars which appeared to be practically undamaged at the end of three years had already shown an expansion at the end of the first winter, and this became more pronounced as time went on. It is my own view that when copings, steps, ballustrades and the like are specified in cast stone for severe climates, then the avoidance of low coefficient aggregates would be very wise. The fact that these spectacular failures do not often occur is no reason for saying that these aggregates have not given any trouble in other cases—there is very good reason to believe that they have.

MR. PEARSON
(In response to a question)

This is the only case in which a cast stone failure from this aggregate was identified as such. I remember that one other job made with this stone was inspected which bore somewhat the same earmarks, but at that time it was in a much less advanced stage of disintegration, and we did not then have any suspicion as to the cause of the trouble. This statement applies also to a number of other cases that had come to our attention, which we felt in some way were tied up to the method of casting in sand molds. Of course, we learned from this experience that

sand casting does not explain all the troubles with cast stone, although I feel quite sure that it is responsible for some of them.

MR. PEARSON
(in response to a question)

These particular steps were ground on the exposed faces; that was to bring out the aggregate pattern. When I examined the stone in March, 1938, there was not much choice between the condition of the vertical faces and the treads; it was somewhat more pronounced on the treads but both risers and treads were in bad shape.

MR. PEARSON
(in response to a question)

The ashlar in the walls was of a similar character, but the exposure was nothing like as severe as that of the steps. There were signs of cracking which might have suggested the beginning of disintegration of a similar type, but I think it would take much longer to develop.

—and in response to a question

The curing practiced in this plant was to cure initially in the sand molds, then by frequent sprinkling until the stones were faced, and thereafter by sprinkling for a considerable time in the yard, depending upon the time available. In this case, of course, sand molds were not used, but frequent sprinkling was continued from the time the steps were removed from the rigid molds.

By F. V. REAGEL and T. F. WILLIS*

Mr. Pearson has brought to the fore a possible factor in concrete durability which has too long been pointed at with suspicion and then neglected. His paper undertakes to explain an early failure of concrete, serving under conditions such that the concrete was apt to be saturated much of the time. From the appearance of the failure, Mr. Pearson suspected that the aggregate might be at least a contributing factor. Freezing and thawing tests "proved conclusively that the marble aggregate caused a rapid failure of the concrete in which it was used." Determination of the thermal expansion of the aggregate showed that the coefficient was relatively small, and over the lower portion of the temperature range was considerably smaller than that of the cement matrix. From the measured thermal expansion and elastic modulus of the aggregate and assumptions as to the thermal coefficient and elastic modulus of the paste, Mr. Pearson makes a "rough estimate" of the internal stress possible under certain conditions and deduces that "it seems very probable.....that the stresses were at times sufficient to produce

*Respectively Engineer of Materials and Chief of Research Division, Bureau of Materials, Missouri State Highway Commission, Jefferson City, Missouri—discussion by letter.

cracks in the matrix. This would have.....afforded opportunity for effective frost action." To confirm his preliminary tests and theory, other aggregates having very small thermal coefficients were incorporated in concrete bars and these subjected to freezing and thawing. As an indication of deterioration, the expansion of the bars after successive cycles of freezing and thawing was measured, and he states that ".....an expansion of .1 to .2 per cent represents practical failure." For the concretes containing aggregates of small thermal coefficient, this condition was attained in from 10 to 60 cycles whereas bars of good concrete (not a part of this series of tests) showed less than .02 per cent expansion in 140 cycles. From the above, Mr. Pearson concludes ".....that the low coefficient of the dolomitic marble in the case described was the prime cause of the failure....."; but that a ....." "factor which no doubt contributed materially to the rapid disintegration" was that the concrete was probably kept more or less saturated during the period in which it was subjected to freezing and thawing.

Some years ago, in connection with observations of a large mileage of concrete pavement in Missouri, it was noted that disintegration, characterized initially by a typical map-cracking, occurred only when a specific kind of coarse aggregate had been used in the concrete. From knowledge of the cracking characteristics of pavements containing various kinds of aggregate, it was thought that the kind causing trouble might have had a coefficient of thermal expansion, or elastic properties, differing considerably from those of the cement paste or mortar matrix. Under the right conditions, this might have caused localized internal stresses sufficient to cause cracking of the matrix. An investigation of the thermal coefficient and modulus of elasticity of various aggregates was initiated. The methods used and some of the results obtained have been previously published.*

In Table C are shown typical results for various rock specimens, all of which were obtained from sources supplying coarse aggregate used in concrete pavements in Missouri. The thermal coefficients were obtained by measuring linear deformations as the temperature of the specimens was increased from 40 to 140 F. Unfortunately, the thermal coefficients were not measured over the lower range (down to −20 F.) which Mr. Pearson thinks may be critical.† However, some of the information is pertinent to his investigation. Fig. B and C show typical curves of the temperature-deformation relations obtained. When the relation was similar to that shown by Fig. B, the only value given in the table is that for the entire range. When the relation was similar to

*T. F. Willis and M. E. DeReus, "Thermal Volume Change and Elasticity of Aggregates and Their Effect on Concrete," *Proceedings, American Society for Testing Materials*, Vol. 39, pp. 919 to 928, 1939.

†Tests are now being made to determine the thermal coefficients of these specimens over the range −30 to + 40 F.

TABLE C

Type of Aggregate	Specimen Number	Linear Thermal Coefficient x 10^{-6}					Modulus of Elasticity x 10^{-6}
		For Complete Range 37 to 140 F. Coefficient	For Lower Portion of Range		For Upper Portion of Range		
			Range	Coefficient	Range	Coefficient	
Chert	1.11	7.1	.....	....	.....	....	18.0
Chert	1.13	7.2	.....	....	.....	....	6.3
Chert	3.12	6.8	.....	....	.....	....	13.5
Chert	3.15	6.9	.....	....	.....	....	4.2
Chert	7.12	6.9	.....	....	.....	....	7.1
Quartzite	3.13	7.0	.....	....	.....	....	10.5
Quartzite	4.11	7.1	.....	....	.....	....	6.5
Quartzite	7.14	7.3	.....	....	.....	....	7.3
Sandstone	1.17	6.7	.....	....	.....	....	4.4
Sandstone	3.11	6.6	.....	....	.....	....	4.2
Basalt	8.12	4.5	.....	....	.....	....	15.5
Granite	14.13	3.7	.....	....	.....	....	8.8
Limestone	21.11	2.9	39–96	1.5	96–140	5.1	....
Limestone	21.12	3.9	38–88	2.2	88–140	5.5	8.6
Limestone	22.12	4.0	38–101	2.8	101–138	6.2	10.3
Limestone	22.14	3.6	39–97	2.9	97–137	3.7	7.5
Limestone	24.12	2.6	37–90	2.2	90–137	3.1	10.0
Limestone	24.14	3.7	37–100	3.5	100–135	4.1	8.0

that shown by Fig. C, values are given for the complete range (slope of AB) and for both the higher and lower portions of the range (slopes of lines AC and BC).

It is at once evident that rocks having temperature-deformation curves similar to that shown in Fig. B have a fairly constant coefficient of thermal expansion over the range tested, and probably over a considerably lower range. On the other hand, rocks having temperature-deformation curves similar to that of Fig. C do not have a constant thermal coefficient over the test range. The coefficient for the upper part of the range is greater than for the lower portion of the range. Determinations by John H. Griffith† of the thermal coefficients of several middle western rocks show similar results for limestones and marbles. It is probable that many of these rocks have much smaller, or even negative thermal coefficients over the temperature ranges discussed by Mr. Pearson. Souder and Hidnert‡ further confirm Mr. Pearson's findings. Several marbles they tested have very small or even negative coefficients over temperature ranges encountered in the northern part of this country during winter. There is, therefore, no doubt that aggregates having small or even negative thermal coefficients occur, and there is evidence that they have been rather widely used in concrete.

As a result of the investigation made of Missouri aggregates, it was found that the most serious disintegration, from the standpoint of both early inception and rapidity of progress, was always associated with the use of an aggregate type having a large thermal coefficient. In fact, as a generality, it can be stated that the concrete pavements, containing those aggregates which probably have the lowest thermal coefficients, have the best service record in this state. This is not in accord with Mr. Pearson's theory, but should be tempered by consideration of several factors:

(a) The severity of service conditions from the standpoint of probable saturation of the concrete. There is no doubt that conditions of exposure for numerous areas of pavements in this state are such that the concrete would be fully as subject to saturation during fall, winter, and spring as the steps described by Mr. Pearson.

(b) The temperature cycles prevalent in this state as compared with those under discussion by Mr. Pearson. The most severe conditions in this state are probably milder than the conditions he describes. Occurrences of air temperatures below zero F. are few, although certain sections of the state have numerous repetitions of 15 to 20 F.

†John H. Griffith, "Thermal Expansion of Typical American Rocks," Table 3, p. 21, Bulletin 128, Iowa Engineering Experiment Station, Ames, Iowa.

‡W. H. Souder and P. Hidnert, "Thermal Expansion of Insulating Materials," Scientific Paper No. 352, National Bureau of Standards, pp. 411-417, (1919).

(c) The fact that the aggregate types which cause trouble in this state are mineralogically quite different from those having low thermal coefficients. The unsatisfactory results with the former probably result primarily from the effect of properties that overshadow any effect of thermal expansion.

Probably the point to be emphasized from the above is that aggregates of low thermal coefficient have been used in concrete, subjected to relatively severe exposure, that has shown satisfactory durability.

As Mr. Pearson stated, the calculation of the internal stress that may be set up in concrete due to difference in the thermal properties of the aggregate and cement paste is practically impossible. However, from approximations he reaches the conclusion that, in the case considered, the stress was probably sufficient to cause cracks in the cement matrix,

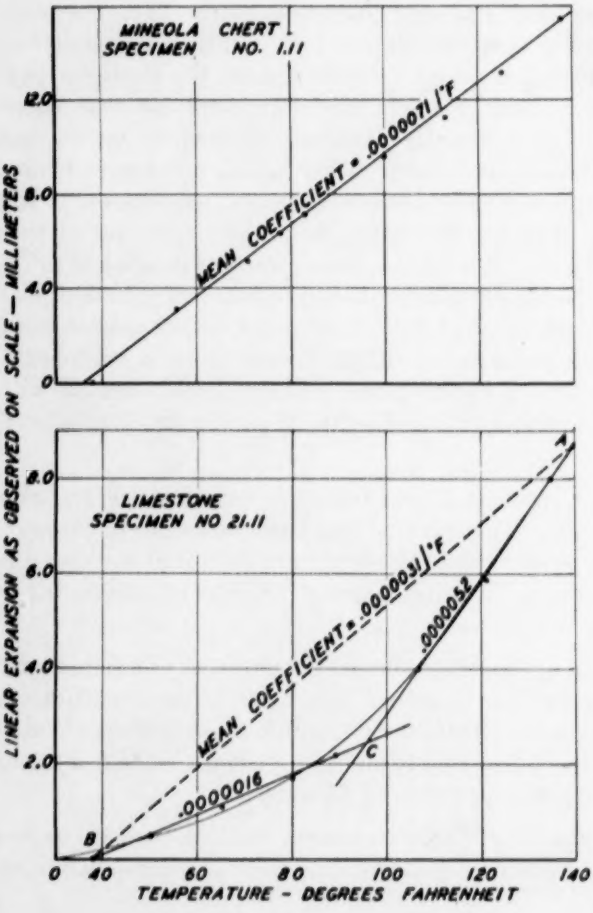
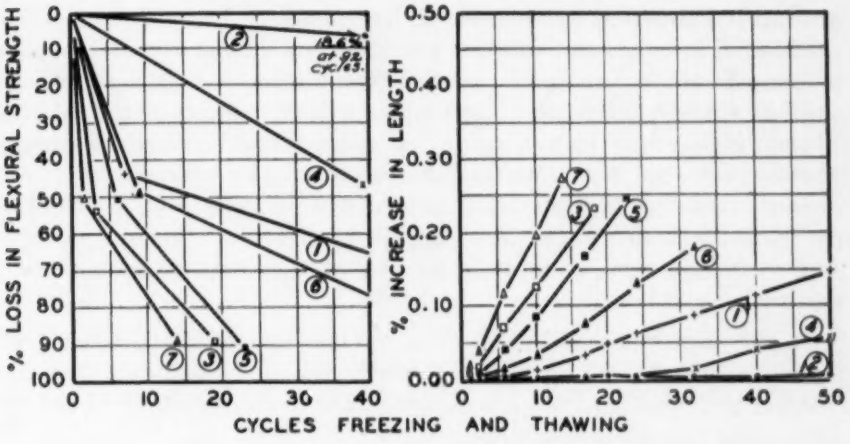
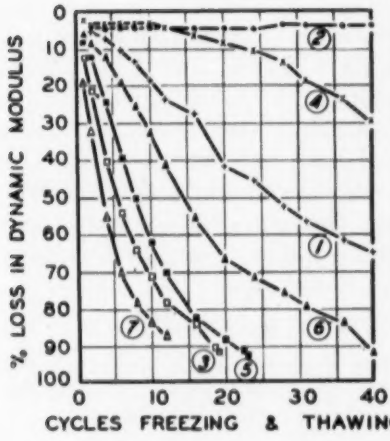


Fig. B (left) C (below) Typical temperature-expansion relations for different rocks.





COARSE AGGREGATE		% ABS.	SLUMP
+	AGGREGATE NO. 1	3.12	5"-4¾"
•	" NO. 2	1.42	2½"-3¾"
□	" NO. 3	3.73	3½"-3¾"
×	" NO. 4	1.81	2"
■	" NO. 5	3.86	3½"-4¾"
▲	" NO. 6	2.59	1⅞"-2"
△	" NO. 7	4.08	2½"

OTHER DATA

PROPORTIONS: (All mixes) 1.00 : 2.88 : 4.15 (By absolute volume.)

WATER CEMENT RATIO: (all mixes) 0.77

CEMENT FACTOR: 1.45

FINE AGGREGATE: Chert-quartz sand. (air dry)

CURING: 7 days in moist-room, 21 days in water.

All coarse aggregate was saturated. All concrete was machine-mixed. Specimens - 3½" x 4½" x 16" prisms.

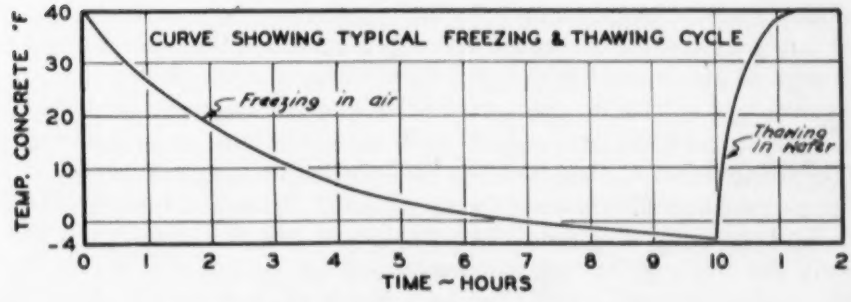


Fig. D—Typical results of freezing and thawing concrete

making it particularly vulnerable to frost action. The factor in such a calculation for which no data are available is the magnitude of the flow or creep of the cement matrix under tensile stress. It is possible that creep of the cement paste could prevent a large portion of the internal stress which would be set up if the cement matrix were completely elastic. Tests in this laboratory of neat cement specimens under compressive stress of 1680 psi. resulted in a flow of .000035 to .000070 ins. per in. in one hour. Information on the creep of neat cement specimens under tensile stress would be valuable to the establishment of Mr. Pearson's hypothesis.

To investigate his theory, Mr. Pearson made concrete specimens containing aggregates of low thermal coefficient and subjected them to freezing and thawing. As a measure of the deterioration of the specimens, their expansion was measured.

In Fig. D the results of a series of tests picked at random from a large number that have been made in this laboratory are shown. There are several points of difference in test procedure which may affect any quantitative comparison with Mr. Pearson's results, but it is thought that comparison of trends is justified. Study of the two upper and the lower left-hand diagrams shows that, if anything, Mr. Pearson's criterion of failure (.1 to .2 per cent expansion) is too conservative. Of the five concretes which were tested until they showed 0.1 per cent expansion, all showed over 65 per cent loss in dynamic modulus and over 60 per cent loss in flexural strength. These values have been verified by a number of other series of tests performed in this laboratory.

As was indicated by the pavement surveys, in the laboratory tests the two concretes (2 and 4) which show the least deterioration are those containing aggregates having the smallest thermal coefficients over the range 40 to 140 F., and each of these aggregates probably has a very small coefficient over the range of 0 to 40 F. On the other hand, the other five concretes, all of which show rather severe deterioration, contain aggregates having about the same relatively large coefficient (6.3 to 7.0 x 10^{-6}) over the range 40 to 140 F. and a temperature-length change relation similar to Fig. B. Comparison of these results with Mr. Pearson's is subject to the qualification that the aggregate groups of low and high coefficient are quite different in their mineralogical and surface characteristics; also, that the "low-coefficient" aggregates may not have as small coefficients as those used by Mr. Pearson. However, these and other similar tests parallel the findings of pavement surveys in this state that, as a rule, concretes containing the aggregate types having the smallest thermal coefficients have shown the greatest durability in service.

One fact prevents acceptance of Mr. Pearson's test results as complete substantiation of his theory of the cause of the disintegration of the concrete steps. In the tests as conducted, any effects of both the differential thermal expansion and the mechanical action of frost are exerted simultaneously toward disintegration of the concrete. It is impossible to determine how much of the measured deterioration, if any, is due to the strains caused by differential expansion. The tests do show a tendency for the concretes, containing the aggregate having the smaller thermal coefficients, to disintegrate most rapidly. However, there are exceptions which must be explained by reference to probable surface characteristics. Moreover, all the specimens that disintegrated at all did so very rapidly. Experience in this laboratory would indicate that comparison of the rates and orders of magnitude of the disintegration of a series of concretes that break down as rapidly as those in Mr. Pearson's tests may not be significant. In other words, repetition of Mr. Pearson's tests might show the relative positions of some of the curves in his Fig. 1 interchanged.

The following is suggested as a test program that might throw additional light on the importance of the role played by the thermal expansion of the aggregate:

Make a series of similar concrete beams using the aggregate in question; cure 14 to 28 days moist.

Dry all the beams in air at room temperature, measuring length change at intervals, until they have practically reached constant weight and length. Divide the series of beams into five sets, each set consisting of the same number (at least three and preferably five) of beams.

Set (A)—Break in flexure at end of the drying period when treatment of Set (B), described below, is started.

Set (B)—Subject these beams to 20 cycles of temperature change, *in air*, from 75 F. to −10 F., measuring length change every few cycles, then break in flexure.

Set (D)—Continue these beams in drying room, measuring length change at intervals, until Set (B) is tested in flexure, at which time test this set in flexure also.

Set (C)—Subject these beams to same temperature change as Set (B). After twenty cycles, place these in water at room temperature an soak for two days. Subject to freezing and thawing test, freezing in air at + 15 F. and thawing in water at 40 F.* Measure length change every few cycles.

*Use of this freezing and thawing cycle would reduce thermal volume change to a minimum at the same time causing, according to tests in this laboratory, almost as much destructive frost action as a lower freezing temperature.

Set (E)—Continue these beams in drying room until Set (C) is put to soak, then treat similarly to Set (C).

Comparison of the test results of Sets (A), (B), and (D) should give some information on whether the differential thermal contraction between aggregate and paste per se, without help from the mechanical action of frost, causes material disintegration of the concrete. It is possible, of course, that the internal strains would cause only incipient cracks which would fail to be reflected by the not-so-delicate flexure test. Also, the concrete subjected to the drying treatment outlined for these sets might be better able to withstand the expected internal strains caused by differential thermal volume change, since the cement matrix of this dry concrete would probably have greater plastic flow, greater strength, and smaller elastic modulus than the saturated concrete Mr. Pearson describes.† Nevertheless, there is sufficient possibility of obtaining direct evidence of the internal stress to warrant the tests.

Comparison of the rate of expansion of Sets (C) and (E) under freezing and thawing should be illuminating. If differential thermal volume change of the paste and aggregate causes sufficient stress to crack the matrix, Set (C) should take up water faster and in greater quantity than Set (E) and under subsequent freezing and thawing should expand much more rapidly. Tests in this laboratory have shown that concrete which has partially deteriorated under freezing and thawing, then been allowed to 'heal' until most of its original strength is regained, will deteriorate at a much more rapid rate under subsequent freezing and thawing than it did initially.

Still another series of tests would give generalized information on the part played in the freezing and thawing test by 'thermal heterogeneity' of concrete. The series would consist of sets of concrete specimens, using glasses of different thermal expansion as coarse aggregate in each set. Vycor glass would provide the aggregate of low thermal coefficient (.0000005 in. per in.) and other glasses are available, the thermal coefficient of which, range from .000002 to .000007 in. per in. per degree F. By using glass as coarse aggregate, the effects of surface texture and absorption, which would be factors in any tests using natural coarse aggregates having different thermal coefficients, would be practically eliminated.

Despite the tenor of part of this discussion, there is no feeling that the information presented invalidates Mr. Pearson's thesis. Even though his evidence like the apparently contradictory information which is

†Raymond E. Davis and G. E. Troxell, "Modulus of Elasticity and Poisson's Ratio for Concrete, and the Influence of Age and Other Factors Upon These Values," *American Society for Testing Materials*, Vol. 29, Part II, (1929), principally pp. 700-701.
Also Herbert J. Gilkey's and Frederik Voght's discussion of above paper, Ibid.

presented herein, is inferential rather than direct, Mr. Pearson's work has resulted in some pertinent indications that the thermal characteristics of the aggregate played a prominent part in the concrete disintegration he describes. However, too often an explanation of one concrete failure is seized on as a likely explanation of many others. Mr. Pearson warns against taking an alarmist view of the possibilities of damage from low coefficient aggregates. One of the principal intents of this discussion is to emphasize this warning by pointing out a case where concretes containing aggregates of low thermal coefficient have given service that is satisfactory, much more so than concretes containing the available aggregates of higher thermal coefficients.

CLOSURE BY MR. PEARSON

In bringing this account of a cast stone failure to the attention of the Institute, the writer realizes full well that his deductions indicate a possible source of trouble in concrete which heretofore has been largely disregarded except in theory. Therefore, the extensive discussion is gratifying, and the fact that some of it indicates skepticism in regard to the conclusions reached does not make it less valuable nor less welcome.

Mr. Meissner has contributed interesting data which, from his close association with extensive concrete researches and field experience, should be given a good deal of weight. But the data in his Table A indicate concretes of such very low resistance to freezing and thawing that many other things need to go into the record beside the tabulation of thermal coefficients. In such comparisons it is very necessary that the concrete be inherently of high resistance to freezing and thawing, this being demonstrated either by complete records, or preferably, by blanks which show the behavior of normal aggregate concrete under the conditions of test. Likewise, the evidence in Mr. Meissner's Table B is rather negative in its relation to the effect of thermal coefficient; it merely emphasizes how completely details of tests must be recorded to enable interpretations to be made. However, this same sort of criticism applies more or less to the writer's own work, for the quantitative effect of high shrinkage of the matrix against a rigid low coefficient aggregate has not been segregated from the over-all effects of freezing and thawing. This will evidently have to be done before a convincing demonstration of the effect of low thermal coefficient aggregates can be made.

It is in this respect that Messrs. Reagel and Willis' discussion is particularly helpful. These gentlemen have rightly emphasized that the effect of creep of the cement matrix on the mitigation of internal stresses can neither be evaluated nor disregarded, and it is to be hoped that they will carry out the program of further tests in the manner they propose.

Also their own field experience is not to be disregarded in its bearing on the problem, as furnishing evidence apparently contradictory to that presented in the paper. But again it seems pertinent to point out that many things other than thermal coefficient affect durability, and that usually there are too many unknowns contributing to net results in the field to permit generalization on the effects of a single adverse factor.

This point seems to be well illustrated in Mr. Hornibrook's account of deterioration in a concrete stadium. Thermal coefficient of aggregate was in this case low enough to have been a contributing factor, but suspicion would attach also to the use of crystalline limestone for both fine and coarse aggregate, to the harshness of the mix, and to the severity of exposure in regard to moisture and temperature. The diagnosis in a case of this sort involves the careful weighing of a number of things that may contribute to disintegration, and usually there is no means of assigning exact weights to the different factors.

Both Mr. Hornibrook and Mr. Swayze have called attention to the fact that calcite crystals have a high positive thermal coefficient along one axis and a smaller negative coefficient perpendicular to this axis. This behavior would seem to have a particularly disruptive effect under extreme temperature alternations, and probably accounts for the phenomenon known as "sugaring," observed in certain types of coarsely crystalline calcites and marbles. However, there is some reason to discount this type of double deformation or volume change as contributing materially to the early failure of concrete. In general, the larger pieces of aggregate in this type of material are agglomerates of crystals, and it seems to be the over-all thermal coefficient of such material that determines the major disruptive stresses. At least, this was presumably the case with the dolomitic marble discussed in the paper, because the effect of 300 cycles of freezing and thawing on 200 pieces of the aggregate was merely to split a few pieces and to produce half a thimbleful of fine grains. It is therefore the writer's present opinion that the over-all coefficient of the aggregate in the vicinity of the freezing point and below is the one of major significance in this problem.

Messrs. Young and Kellam have used 4-in. cube specimens for thermal coefficient determinations, which seem to have the merit of economy, as well as the advantages of slow temperature change while measurements are being made in three mutually perpendicular directions. The only disadvantages seem to be the necessarily long time required for such specimens to arrive at temperature equilibrium, and the very careful manipulation procedure required for accurate calipering.

In attempting finally to weigh the evidence for and against the theory of failure set forth in the paper, and bearing in mind that the concrete

in this case was of high quality by all the usual criteria for well made concrete, the writer sees clearly only two major factors that could account for the failure, (1) a low thermal coefficient aggregate which caused high internal stresses in cold weather, (2) an unusually severe exposure which at the worst might be considered comparable to conditions imposed in laboratory freezing and thawing tests. The relative effects of these two factors cannot be determined quantitatively from the data available—it is merely known that the test bars were visibly succumbing to frost action after some 20 cycles of freezing and thawing, and if Messrs. Reagel and Willis' implied end point of 0.1 per cent expansion were accepted, then this apparently high quality concrete failed at about 16 cycles. *Normal concrete just does not fail that early* in freezing and thawing tests of this sort (freezing immersed at zero F., thawing immersed at 70 F.), and therefore it seems necessary to fall back on the effect of low thermal coefficient as the correct explanation. Perhaps the apparently conflicting evidence brought out in the discussion may not conflict if one imagines the internal stress set up by the restraint of the aggregate at low temperatures to act as a sort of primer for severe frost action, possibly through formation of sub-microscopic cracks or strains, which may have relatively little effect in the absence of icing conditions. However, the actual mechanism of failure in cases of this sort ought to be made clear by further studies. There seem to be no insurmountable difficulties in the way of getting all the information needed.

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Bond Stress in Concrete Pull-Out Specimens*

By DAVID WATSTEIN†
Member American Concrete Institute

SYNOPSIS

The purpose of this investigation was to determine the distribution of bond stress in pull-out specimens. Thirty-six 6- by 18-in. cylindrical concrete pull-out specimens containing ¾-in. diameter round bars were tested. There were 4 types of deformed bars and 2 types of plain bars (hot and cold rolled). The bond stress at the loaded end of all bars except the cold rolled bar, varied approximately linearly with the applied load. The bond stress at the loaded end of the deformed bars was considerably greater than that at the free end.

I. INTRODUCTION

The distribution of bond stress was determined for six types of steel bars embedded in 6 x 18-in. concrete pull-out specimens. One series of 18 pull-out specimens was designed to permit measurement of strain at the quarter points along the embedded bars. In another series of 18 specimens only the overall elongations of the embedded bars were measured. Concrete of the same proportions was used throughout and all specimens were cured in moist air and tested at the age of 28 days.

II. MATERIALS

1. Steel bars

The six types of steel bars tested are shown in Fig. 1. The plain bars A and B (cold and hot rolled) were obtained from the stock of the machine shop of the Bureau, while the deformed bars of intermediate grade (C to F) were purchased to conform with Federal Specification QQ-B-71a, grade 2.

The diameters of the plain bars (A and B) were determined by micrometer calipers, while those of the deformed bars were determined from the length and weight measurements of the bars. The yield point of the steel bars was determined in a 100,000 lb. capacity testing machine of the beam and poise type by observing the drop of the beam.

*Received by the Institute, Nov. 30, 1940.
†Asst. Materials Engineer, National Bureau of Standards, Washington, D. C.

The physical properties of the steel bars are given in Table 1.

TABLE 1.—PHYSICAL PROPERTIES OF THE STEEL BARS

Type of Bar	Diameter in In.	Yield Point p.s.i.	Tensile Strength p.s.i.	Modulus of Elasticity 10^6 p.s.i.
A	.750	.....	93,300	29.6
B	.756	50,100	88,400	29.5
C	.747	43,300	74,000	28.8
D	.749	47,900	80,300	29.2
E	.739	45,200	75,100	29.9
F	.742	48,500	84,400	29.7

2. Concrete

The concrete was proportioned by weight in the ratio of 1:2.8:3.2. The portland cement met the requirements of Federal Specification SS-C-191a for Portland Cement. The fine aggregate was Potomac River concrete sand. (Table 2.)

TABLE 2—GRADING OF POTOMAC RIVER CONCRETE SAND

U. S. Standard Sieve No.	Percentage Passing (by Weight)
4	98.3
8	87.3
16	77.1
30	60.0
50	21.6
100	2.3

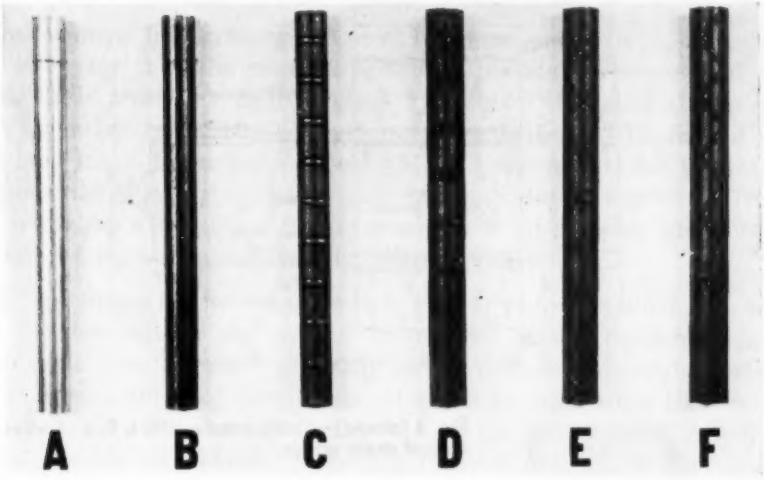
The coarse aggregate was Potomac River gravel and consisted of two fractions: 37.5 per cent was No. 4 to ⅜ in. gravel and the remainder ranged from ⅜ to ¾ in. The concrete contained about 5.7 bags of cement per cu. yd.; the water-cement ratio by weight was 0.65, and the slump approximately 2 in.

The average compressive strength of 6 x 12-in. cylinders of the concrete cured in moist air, was 4310 psi, and the modulus of elasticity was 4,010,000 psi at 28 days.

III. PULL-OUT SPECIMENS

1. Description of test specimens

Two series of 6 x 18-in. cylindrical pull-out specimens containing ¾ in. round bars were tested. The specimens of the first series were designed for measurement of distribution of stress along the bar by providing twelve ¼-in. openings arranged along two diametrically opposite sides of the bar (Fig. 2 and 3). These openings exposed the embedded steel bar at gauge points and permitted measurement of tensile strain at the quarter points of the bars.



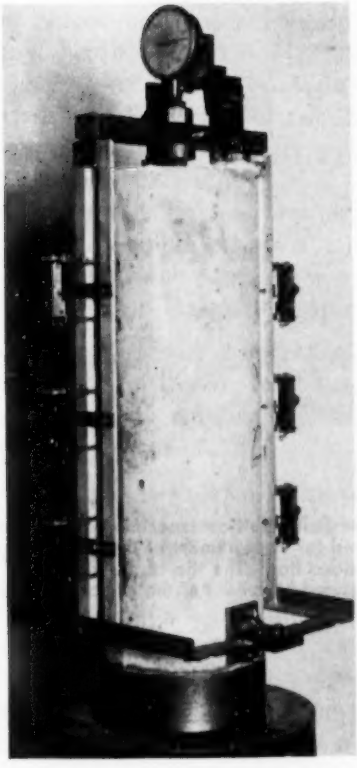


Fig. 1 (above)—Reinforcing bars. All bars have a nominal diameter of ¾ in. Bar A is a smooth cold rolled bar; bar B is a hot rolled bar having a mill scale surface; bars C, D, E and F are deformed bars of intermediate grade.

Fig. 2 (left)—Perforated pull - out specimen with strain transfers mounted at quarter points along diametrically opposite sides of the embedded bar.

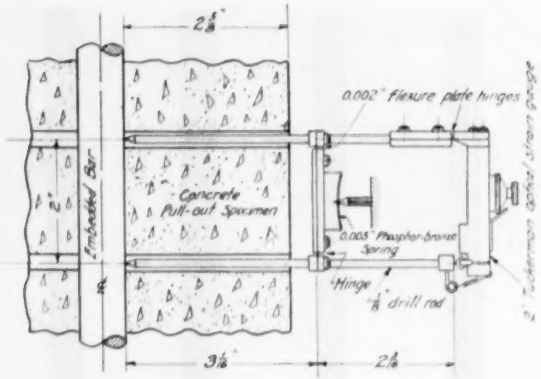


Fig. 3 (above)—Strain transfer with a 2-in. Tuckerman optical strain gauge.

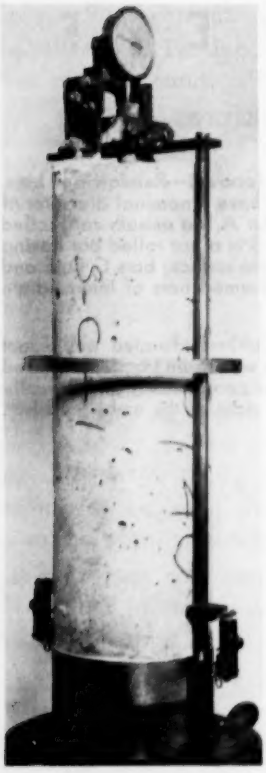


Fig. 4 (left)—Solid pull-out specimen with extensometer mounted for measurement of overall elongation of the embedded bar. The slip of the free end of the bar was measured by means of the micrometer dial.

The openings for exposing the steel bar were made by ¼-in. round steel rods cast into the concrete. During placing of concrete in the molds, these rods were held in place by slots in the mold at one end while the other ends fitted into 1/16 in. holes drilled in the side of the embedded bar. These holes served also as gauge points for the strain transfers which were inserted into the openings during the test. These rods remained undisturbed in the specimen until the day previous to testing, and then were removed by twisting with pliers.

The specimens of the second series (Fig. 4) were cast solid and were used for determining the overall elongations of the embedded bars. These data were intended primarily to provide an independent check on the measurement of distribution of stress by comparing the elongations observed in the solid specimens with the values computed from known stresses and elastic properties of steel bars in the perforated specimens.

2. Strain gauges

Tension in the embedded bars was measured with 2-in. Tuckerman optical gauges mounted on strain transfers which reduced the measured deformations to about two-thirds of the actual value. The strain transfer (Fig. 3) consisted of two ⅛-in. diameter pointed drill rods fastened to a crosspiece by means of two flexure plate hinges; the pointed ends were seated in 1/16 in. diameter gauge holes in the side of the steel bar, while the other ends supported the Tuckerman gauge. Two such strain transfers mounted for calibration on bar F are shown in Fig. 5; the transfers were supported by a stress-free frame which was clamped to the steel bar at the top and was maintained in alignment by flexure plates attached to the pull-out specimen at the bottom.

The strain transfers were calibrated in pairs by observing the simultaneous values of tension in the calibration bar and the readings of the two gauges. The strain transfers were similarly paired when used on the pull-out specimens.

The extensometers used in measuring the overall elongations of embedded bars in the solid specimens consisted of two 2-in. Tuckerman gauges mounted on extension bars (Fig. 4); during the test, the extension bars were attached to the free end of the bar by means of flexure plate hinges, while ¼-in. diameter pointed drill rods transferred the strain in the bar to the Tuckerman gauges in the ratio of 3.42 to 1. The extensometer was calibrated against a pair of Tuckerman gauges which were mounted on the calibrating device and measured the movement imparted to the extensometer. The dimensions and operation of the extensometers are shown in Fig. 6.

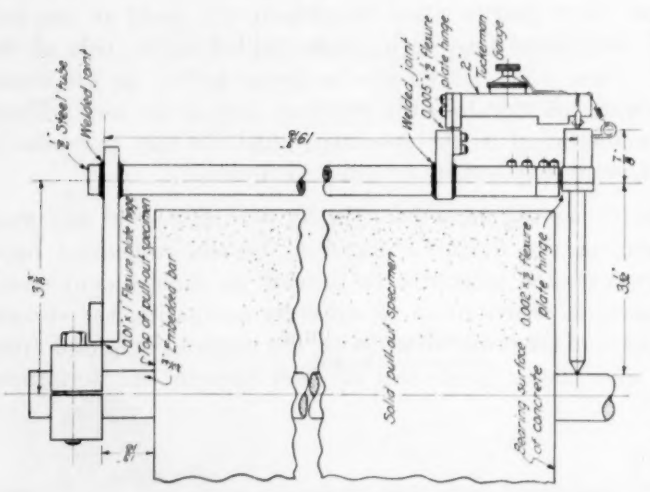
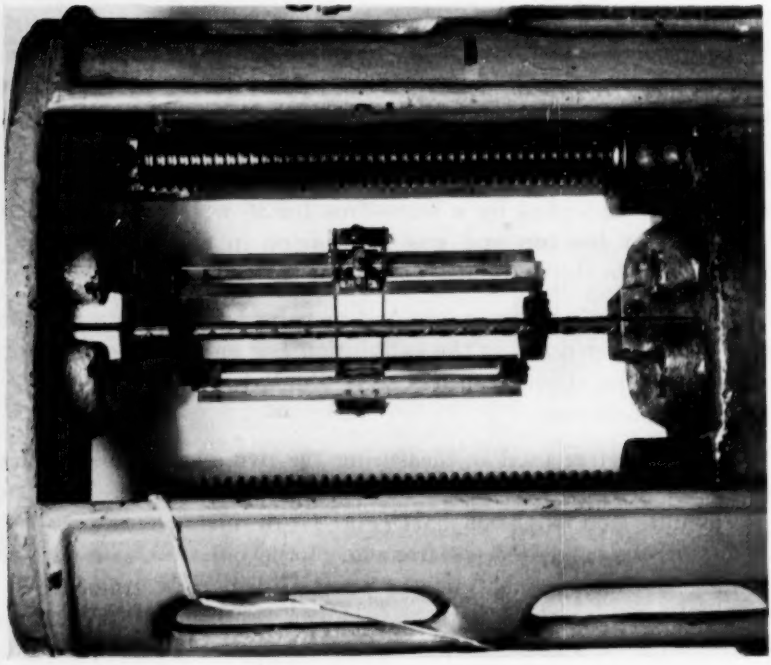


Fig. 5 (left)—Strain transfer and gauge mounted for a calibration against tension in bar F.

Fig. 6 (above)—Extensometer for measuring the overall elongation of the embedded bar.



3. Method of testing

(a) *Perforated pull-out specimens*—The lower faces of the pull-out specimens were capped with a neat mortar of plaster of paris and the caps were allowed to harden one day before test. The plaster-capped faces of the pull-out specimens were supported on a spherical bearing block having a 1-in. hole in the center, and were tested in a 100,000-lb. capacity testing machine of the beam and poise type. An initial load of 200 to 300 lb. was applied and the initial readings of the strain gauges were noted. The load was applied thereafter in increments of 1000 or 2000 lb., depending on the expected maximum load. The initial slip of the free end of the bar was observed by means of 0.0001-in. micrometer dial supported on the top of the pull-out specimens adjacent to the free end of the bar. Strain readings were not observed after the applied load had passed its maximum value, since the strain transfers were found to have appreciable backlash and did not give a correct measure of the decreasing tension in the bar.

(b) *Solid pull-out specimens.*—The solid specimens were capped and tested in a manner similar to that described, except that the specimen rested on two circular segments attached to the face of the spherical bearing block, the segments being spaced 1 in. apart to provide a gap for inserting the lower arm of the extensometers; the extensometer was engaged at two gauge holes drilled in diametrically opposite points $\frac{3}{8}$ in. below the bearing surface of the pull-out specimen.

The initial reading of the extensometer was noted at a load of about 50 lb. and the load was applied thereafter in increments as described.

IV. RESULTS AND DISCUSSION

1. Perforated pull-out specimens

Each set of curves (Fig. 7 and 8) represents the average data obtained with the three specimens containing bars of the same type. The maximum loads indicated on these curves are approximately those producing failure of the pull-out specimens. The loads at first slip and maximum loads for individual specimens are given in Table 3.

The distribution of tensile stress in the embedded bars is shown in Fig. 7; the values of tension at the quarter points of the bars are those computed from the strain transfer readings, while the tension at a distance of 18 in. from the free end is the load applied to the specimen by the testing machine. The curves in Fig. 8 show the distribution of bond stresses, which were estimated from the slope of curves in Fig. 7 and the perimeters of the embedded bars. It is emphasized that the error involved in fitting smooth curves to the experimentally obtained points of Fig. 7 was further magnified in measuring the slopes of these

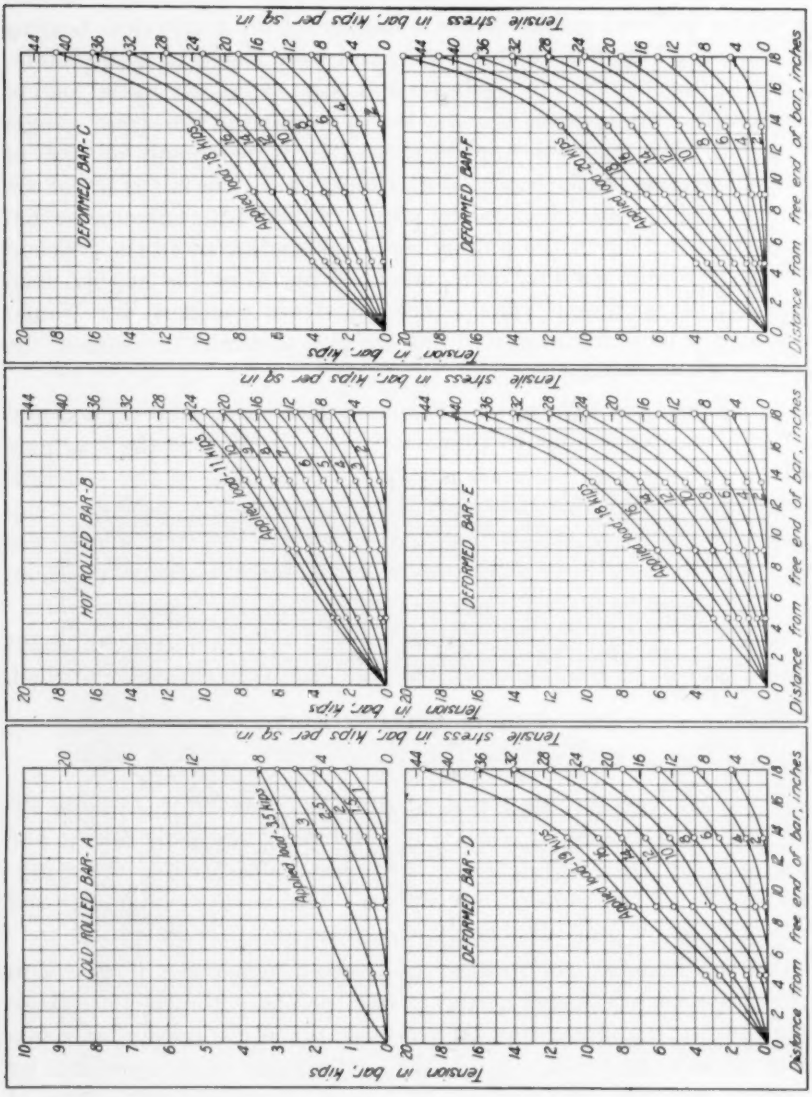


Fig. 7—Distribution of tensile stress in bars embedded in pull-out specimens.

TABLE 3.—LOADS AT FIRST SLIP AND MAXIMUM LOADS IN PULL-OUT SPECIMENS

Bars	Perforated Specimens				Solid Specimens			
	1	2	3	Ave.	1	2	3	Ave.
	Loads at first slip, lb.(1)							
A	3010	2940	3500	3150	3040	3090	—	3070
B	8600	7250	—	7920	11100	11150	10900	11050
C	10450	9020	15350	11670	7580	9130	11800	9500
D	14720	—	12120	13420	15700	16060	17420	16390
E	14950	11750	13020	13240	12330	8130	11780	10750
F	16570	8500	12900	12660	15700	15000	—	15350
	Maximum loads, lb.							
A	3600	3400	3500	3500	3950	3090	4360	3800
B	11200	11400	10660	11090	15660	14500	14770	14980
C	19040	18330	18900	18760	17240	18000	17400	17550
D	21000	20850	21150	21000	20160	20100	20360	20210
E	19600	19520	19360	19490	18920	19000	19480	19130
F	21200	28060	21350	23540	19860	19900	20600	20120

(1) These loads were estimated for a slip of 0.0001 in. at the free end of the bar from plotted load-slip curves, except where the load corresponding to this value of slip was observed during the test.
Note: Bars A and B failed in bond, while all other bars failed by yielding in tension.

curves for estimating the bond stresses shown in Fig. 8; this is particularly true for the relatively steep portions of the curves in Fig. 7 for the deformed bars.

For the smooth cold rolled bar A, the bond stress at the loaded end of the bar increased with the applied load up to a load of 2000 lb. As the applied load was increased further, the bond stress decreased at the loaded end. The bond stress at the free end increased rapidly, from zero stress, as the load increased from 3,000 to 3,500 lb., until, for the maximum applied load of 3,500 lb. the greatest bond stress of 150 psi was developed at the free end. A similar shifting of the location of the maximum bond stress was indicated by the data of Gilkey *et al* for plain bars.* The results obtained by Glanville† for smooth tubes embedded in pull-out specimens were in accord with the observation that the greatest bond stress for the smooth cold-rolled bar A was developed at the free end for the maximum applied load.

For the hot rolled bar B, the bond stresses at both ends of the pull-out specimen tended to become equal as the applied loads approached the maximum value.

There was some similarity among the bond stress distribution curves for the four deformed bars C to F, at loads above those producing first

*Proceedings, Highway Research Board, Vol. 18 (1938) p. 114-129.
†Bldg. Res. Techn. Paper No. 10, Dept. of Scient. and Ind. Res., London (1930).

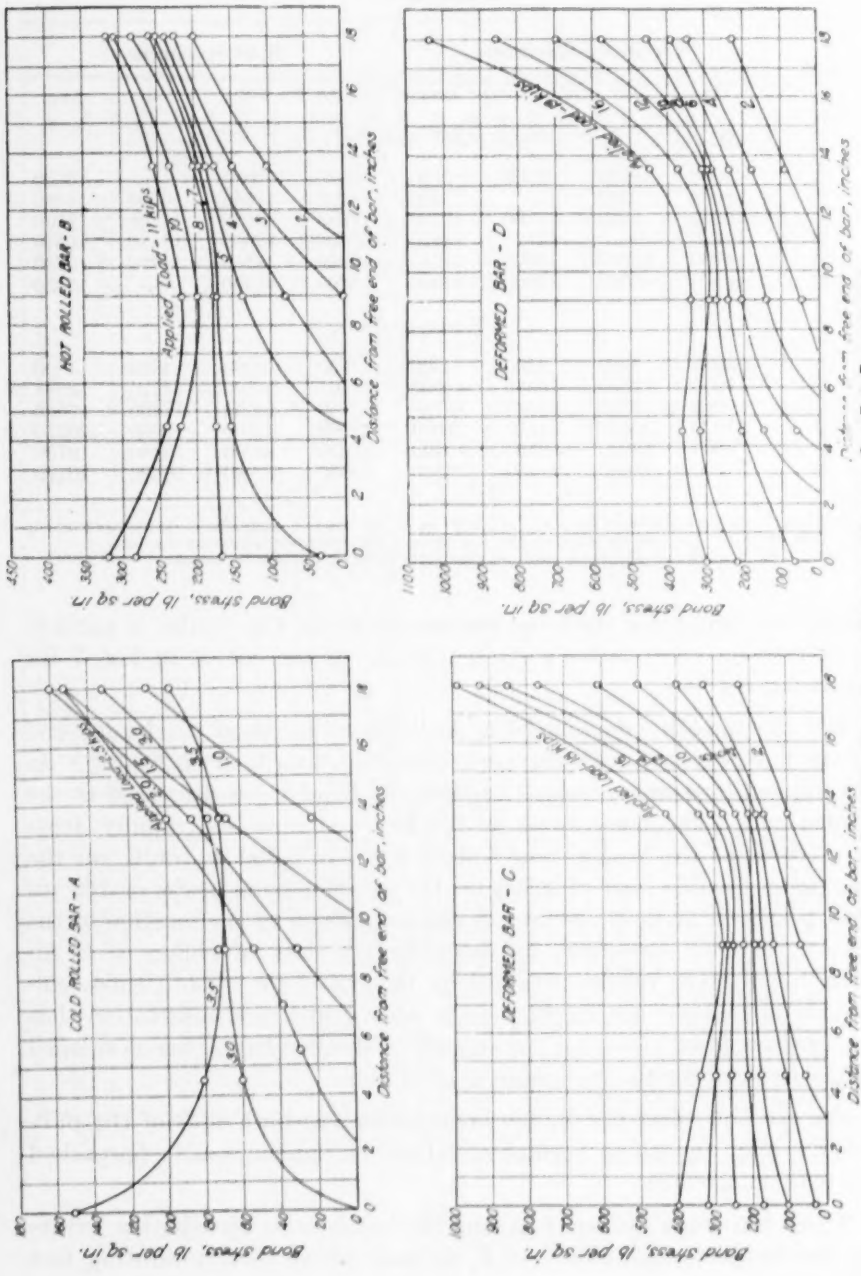

HOT ROLLED BAR - B
DEFORMED BAR - D
COLD ROLLED BAR - A
DEFORMED BAR - C
Bond stress, lb per sq in.
Distance from free end of bar, inches

Fig. 8—Distribution of bond stress in pull-out specimens—See E & F, next page.

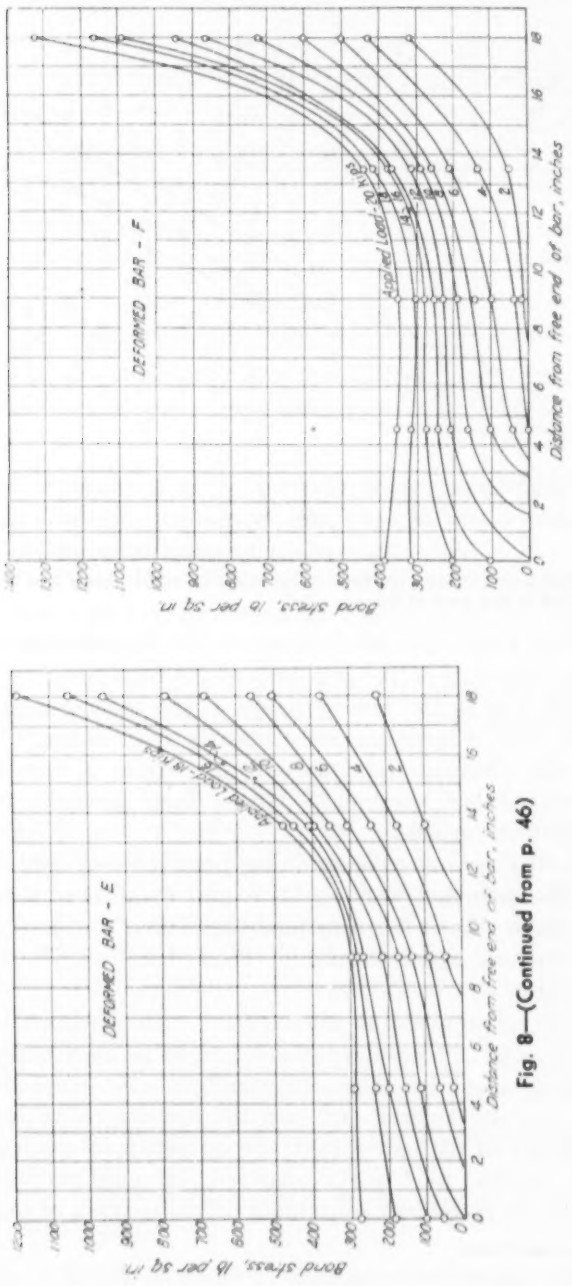

DEFORMED BAR - F
Applied Load, 20 kips
Bond stress, lb per sq in.
Distance from free end of bar, inches
DEFORMED BAR - E
Applied Load, 18 kips
Bond stress, lb per sq in.
Distance from free end of bar, inches

Fig. 8—(Continued from p. 46)

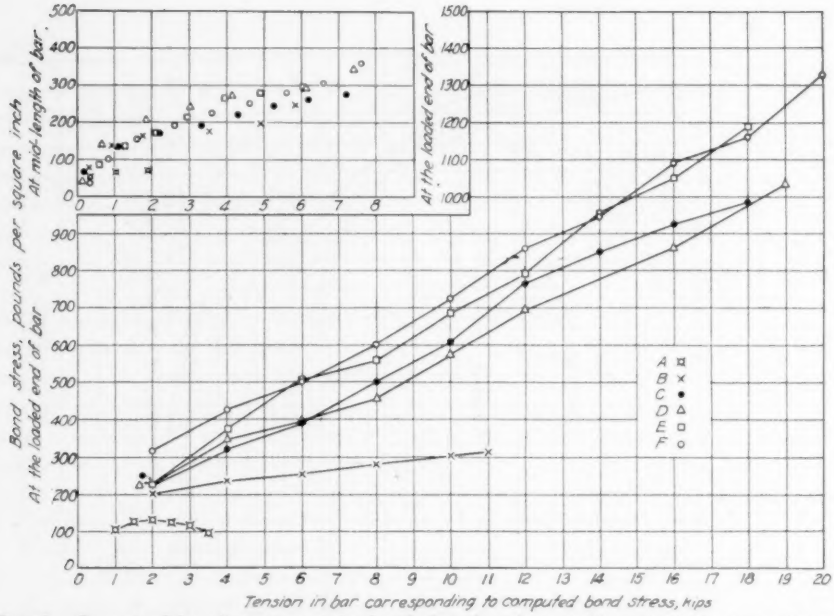


Fig. 9—Computed bond stresses at the loaded end and at mid-length of the bars plotted against the tension in the bars at those points.

slip. For these loads, the bond stress at the loaded ends was considerably higher than that at the free ends of the bars.

As shown in Fig. 9, the bond stresses at the loaded ends of all bars except bar A, varied approximately linearly with values of tension at the loaded ends. Similar data plotted for sections of bars at the mid-length of the pull-out specimens showed more irregularity than the data of the previous group.

The perforated specimens containing plain bars A and B failed in bond, while the deformed bars C, D, E and F failed in tension during the pull-out tests. Since the deformed bars failed in tension, the maximum bond stresses characteristic of these bars were not developed during these tests.

The load-slip relation was determined for each specimen tested (data not presented), and the results were found to be in accord with comparable data obtained by other investigators. No consistent relation between the slip at the free end and the corresponding bond stresses was found for any of the bars tested; the discrepancies among individual specimens of one kind were of the same order of magnitude as the differences observed for specimens containing different types of bars.

2. Solid pull-out specimens

The average observed elongations of the bars in solid specimens for each of the six types of bars are shown plotted against the correspond-

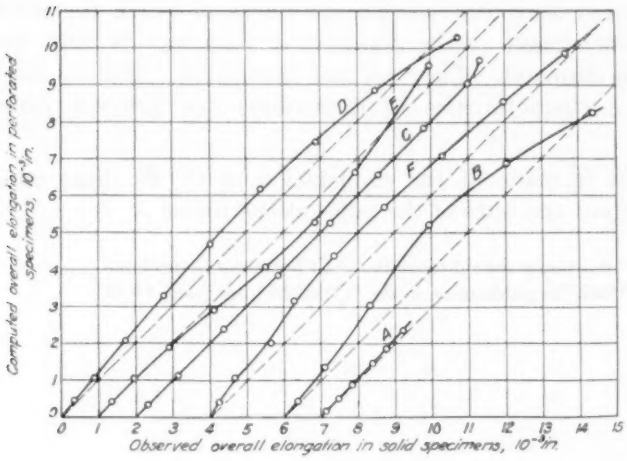

Computed overall elongation in perforated specimens, 10^{-3} in.
Observed overall elongation in solid specimens, 10^{-3} in.
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15
0 1 2 3 4 5 6 7 8 9 10 11
A
B
C
D
E
F

Fig. 10 — Comparison of observed and computed overall elongations in solid and perforated pull - out specimens for corresponding loads.

ing computed elongations in the perforated specimens in Fig. 10. The elongations of bars in perforated pull-out specimens were computed from the measured areas under the stress distribution curves shown in Fig. 7. With the exception of the deformed bar E, the computed elongations were either equal to, or somewhat greater than, the corresponding observed elongations.

The manner of failure of the solid pull-out specimens was the same as that described for corresponding bars in the perforated pull-out specimens.

V. SUMMARY

1. The bond stresses developed at the loaded ends of the deformed bars were considerably greater than those at their free ends for all loads; for the hot rolled bars, the bond stresses at both ends of the bar tended to become equal as the applied loads approached the maximum values, while for the cold rolled bars the bond stress was greatest at the free end and decreased at the loaded end as the load reached its maximum value.

2. The bond stress at the loaded ends of all bars except the smooth cold rolled bar A, varied approximately linearly with the applied load.

3. With the exception of the deformed bar E, the computed elongations of bars in perforated pull-out specimens were equal to or somewhat greater than the observed elongations in the solid pull-out specimens, for corresponding loads.

4. The full values of the bond strengths were not developed for the deformed bars C to F, which failed in tension during the pull-out tests.

5. The results cited were observed for specimens in which the ratio of the length of embedment to the diameter of bars was 24, and the bars were ¾ in. in diameter. They are not necessarily valid for steel bars whose surface characteristics or dimensions are different from those described.

Acknowledgement is made of the cooperation of C. W. Ross who assisted in carrying out the tests of the pull-out specimens.

Discussion of this paper should reach the ACI Secretary in triplicate by Mar. 1, 1942, for publication in the JOURNAL for June, 1942

A part of PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE Vol. 38

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Vol. 13 No. 6 7400 SECOND BOULEVARD, DETROIT, MICHIGAN June 1942

Discussion of a paper by David Watstein:

Bond Stress in Concrete Pull-Out Specimens*

By BENGT FRIBERG†

Credit is due the author for carefully prepared and executed tests. The Data are a valuable addition to the understanding of the bond behavior, a problem still requiring much fundamental knowledge.

The greatest hindrance to accurate interpretation of bond phenomena has been the lack of conformity between the pull-out test and conditions existing in critical structures. May these tests help to eliminate conventional pull-out test for research purposes.

For loads near ultimate, bond stresses along the deformed bar specimens varied over 30 per cent of the bond stress at mid-point. The concrete compression near the bearing block clearly influenced the tests sufficiently to preclude accurate comparison between the different types of deformed bars. This influence being excessive for 18-in. embedment, it would be greater still for common, shorter pull-out specimens.

The tests illustrate that characterization of bond performance as load for first slip at the unloaded end of a pull-out specimen is an indefinite evaluation of bar performance. Data for short pull-out specimens indicate first slips at substantially the same load for plain and deformed bars. In these tests the loads at first slip were for deformed bars from 108 to 155 per cent of the load for plain bar. The ultimate loads for the deformed bar ranged from 139 to 168 per cent of the plain bar load.

In addition to a critical exposure on the pull-out test for research purposes, the paper contains information from which some characteristics of bond behavior may be deduced, at least for the center half of embedment where the bearing block influence is not excessive. The

*ACI JOURNAL, Sept. 1941; *Proceedings* V. 38, p. 37.
†Laclede Steel Co., St. Louis, Mo. Now, Senior Engineer (Civil), Corps of Engineers, St. Louis, Mo.

data in the paper on that subject are limited to the small diagram in Fig. 9 of the paper.

The author has related the bond stress to tensile stress in the bar. Actually, bond development being a surface phenomenon, it should be related to the relative change in position of points at that boundary line, as represented by longitudinal deformation of the steel cross-section. The writer has plotted these data in Fig. A. The longitudinal deformations are represented by the areas under the author's steel stress curves from point of zero stress. Stress at the three quarter points only have been studied except for the cold rolled bar which is less influenced by bearing block restraint.

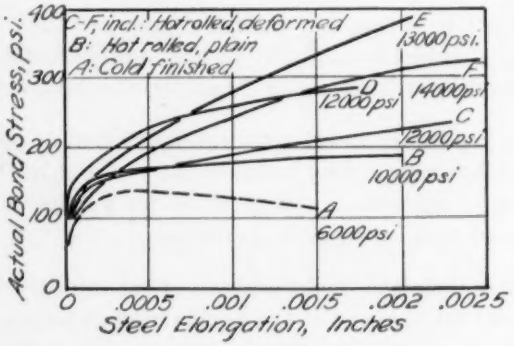

Fig. A—Bond between concrete in compression and steel in tension related to longitudinal steel deformation for hot rolled deformed and plain bars and cold rolled bars

The graph shows a minimum bond resistance, uniform for all the bars, of about 150 psi, which is reached before any substantial longitudinal deformation occurs. Thereafter the bond development is greatly influenced by the bar characteristics.

The value of steel elongation does not represent exactly the relative movement between a steel cross-section and adjacent concrete. This deflection may be 15 to 20 per cent greater than the steel elongation for these specimens. If longitudinal deflections at the unloaded end of the bars had been included, the longitudinal deflection data could have been extended up to and beyond customary steel working stresses. These may be estimated to exist in the deformed bars at steel elongations of .004 in. or more.

Similar information obtained for other concrete strengths and for bars encased in concrete in tension could be expected to yield important information of value in concrete design.

Other literature references indicate that the minimum bond stress for zero movement of 150 psi may recur for conditions different from

these tests. The 150 psi could in that case be considered as a bond yield point. Bond working stresses may then not need to be limited to greatly below that value, considering the great additional safety in bar deformations.

The greatest possible conservation of steel is now of the utmost importance. Full advantage should be taken of increased working stresses wherever increased material knowledge, conditions of loading or occupancy justify a temporary departure from established safety standards.

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Temperature Effects Near Concrete Surfaces as Affected by Heat Liberation of Cement*

By LOUIS R. FORBRICH†
Member American Concrete Institute

SYNOPSIS

This paper attempts to show by computation temperature changes and temperature distributions near formed surfaces of some typical concrete structures before and after removal of forms. The computations showed that near the surfaces temperature rises of 25 to 45 F. may be expected in 1½ to 6 ft. concrete sections. Removal of forms as early as possible before the maximum temperatures occur, or, if this is not practical, removal of forms as late as possible, is indicated as a means of reducing the danger of cracking due to thermal effects. The use of an additive material for reducing early heat liberation of the cement may be effective in reducing temperature rise and cracking tendency of concrete.

INTRODUCTION

Since cracking of concrete structures is attributable to a considerable extent to temperature change, particularly to the drop in temperature from the maximum produced by the heat of hydration of the cement, to the ultimate stable temperature, control of these temperature effects assumes importance. The heat evolved and the temperatures attained depend on many factors, such as type of cement, mix design, initial temperature of materials, size of structure and many others. These factors have already received considerable attention with respect to interior temperatures of large structures, but only in a few instances have temperature effects near surfaces been given consideration.

In a paper by Carlson‡ in 1938 on temperatures and stresses in mass concrete it was suggested that temperature conditions at surfaces may have greater significance with respect to thermal cracking than temperature conditions in the interior of a mass of concrete. It was pointed out

*Received by the Institute April 25, 1940.
†Bessemer Limestone & Cement Co., Youngstown, Ohio; formerly with Master Builders Research Laboratory, Cleveland.
‡Roy W. Carlson: Temperatures and Stresses in Mass Concrete. ACI JOURNAL, March-April, 1938; *Proceedings* V. 34, p. 497.

that because the interior ordinarily cools gradually, allowing plastic flow to relieve stress, the concrete is capable of withstanding relatively large temperature drops without cracking. At surfaces, on the other hand, the concrete is usually subjected to rapid cooling, while the interior is still warming, and therefore, cracking may be more readily initiated. When surfaces are covered by thick wooden forms, which act as insulators to retard the dissipation of heat, this condition may become critical. For example, when concrete is placed according to modern practice the surface immediately inside the forms usually has its maximum temperature rise after about 2 or 3 days. If the forms are removed at this time, and they frequently are, the surface may cool rapidly and cause cracking. Thus it seems that if temperature changes at surfaces can be controlled so as to prevent cracks from starting, then temperature effects in the interior would probably not cause cracking. However, it is not suggested that interior temperatures can be considered lightly; they do have an important bearing on the cracking problem, since they affect the ultimate size of the cracks and the extension of the cracks from the surface to the interior, although they may not be the initial cause of cracking.

In view of the present interest in temperature effects near surfaces especially beneath wooden forms, it seems desirable to study some of the pertinent factors involved. Because temperatures of formed surfaces are mostly affected by the heat evolved during early periods of hydration this paper will be concerned primarily with temperature effects up to 7 days for some typical structures. These will include not only relatively thick sections such as are usually considered as mass concrete but also thinner sections not usually so regarded. A section of the paper will be devoted to a consideration of the effect of an additive material called HR-20*, which has been shown in a previous paper,† to reduce the early heat liberation of cements.

Because temperatures in concrete are affected by so many variables which are not easily controlled, it would be difficult to investigate this subject experimentally or by studying concrete already in service. Therefore, the effects of the variables considered in this paper will be shown by computation, since it has been shown that computed temperatures check closely with observed temperatures under known and controlled conditions. For concrete containing HR-20, experiments have shown that the actual temperature rise is not quantitatively in accord with that computed from data obtained on neat cements, as illustrated

*Composition of HR-20:

	Parts by wt.
Calcium Lignin Sulphonate	0.300
Ortho-hydroxy Benzoic Acid	.075
Fly Ash	.625

†L. R. Forbrich: The Effects of Various Reagents on the Heat Liberation Characteristics of Portland Cement, ACI JOURNAL, Nov. 1940; *Proc.* Vol. 37, p. 161.

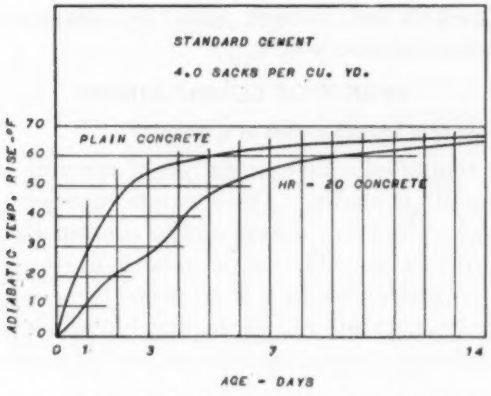


Fig. 1—Computed adiabatic time-temperature curves

below, so that the computations relating to HR-20 concrete can only be considered to indicate trends.

METHODS OF COMPUTATION

The method of computation of temperatures was the simplified method developed by Carlson*. The thermal properties of the plain and HR-20 concretes were assumed to be the same. The thermal conductivity and diffusivity of the concrete and wood used in the computations were as follows:

	Wood	*Concrete*
Thermal Conductivity—k.....	1.80	36.0 BTU per ft. day deg. F.
Diffusivity—h^2.....	0.11	1.0 sq. ft. per day

For the purpose of simplicity, the initial temperature of the concrete, and the temperature of rock, wood, and ambient atmosphere were assumed to be constant at 0°F. Thus an initial concrete temperature of 0°F., may actually mean a temperature of 60 or 70F., as the case may be. Temperature rises, therefore, represent the temperature rise above the initial temperature. For the computation of the temperature distribution within a monolith it was assumed that heat flows only in the vertical direction, while for the computation near surfaces or in walls, it was assumed that the heights of the sections were infinite and that heat flowed only in the horizontal direction toward the forms.

The adiabatic time-temperature curves for concrete containing 4 sacks of a standard (normal) portland cement per cubic yard shown in Fig. 1 and for concrete containing 5½ sacks cement (not shown) were the basis of all the computations of temperature rise. These curves which represent the temperature rise of the concrete if no heat were lost, were computed

*Roy W. Carlson: A Simple Method for the Computation of Temperatures in Concrete Structures. ACI JOURNAL, Nov.-Dec., 1937; *Proc.* Vol. 34, p. 89.

from data obtained on neat cement pastes by means of the conduction and heat of solution calorimeters.*

RESULTS OF COMPUTATIONS

Temperature variations near vertical surface of a monolith

In Fig. 2 the computed temperature rise of concrete near the formed surface of a monolith is shown. The concrete represented was assumed to be placed in 5 ft. lifts every 5 days and to contain 4 sacks of standard (normal) cement per cu. yd. The curves in both diagrams, are for the first lift covered on the vertical side by a 0.2 ft. wood form and were computed from the average temperature rise of the first lift which in turn was computed from the adiabatic time, temperature curves shown in Fig. 1.

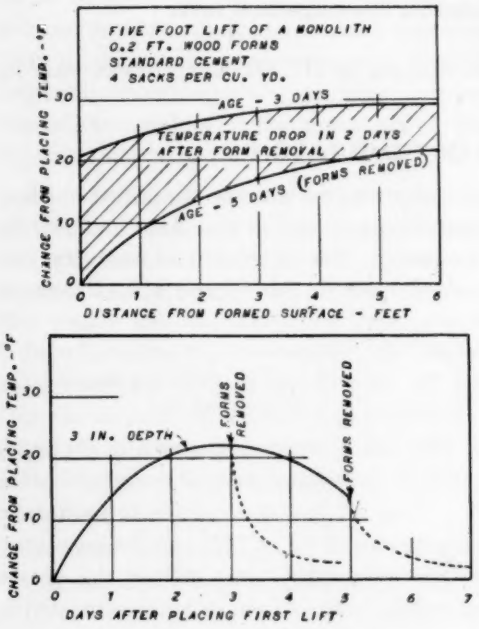

FIVE FOOT LIFT OF A MONOLITH
0.2 FT. WOOD FORMS
STANDARD CEMENT
4 SACKS PER CU. YD.
AGE - 3 DAYS
TEMPERATURE DROP IN 2 DAYS AFTER FORM REMOVAL
AGE - 5 DAYS (FORMS REMOVED)
CHANGE FROM PLACING TEMP. - °F
DISTANCE FROM FORMED SURFACE - FEET
3 IN. DEPTH
FORMS REMOVED
FORMS REMOVED
DAYS AFTER PLACING FIRST LIFT

Fig. 2—Temperature changes and temperature distribution in a 5-ft. lift of a monolith showing the influence of form removal.

In the lower diagram of Fig. 2 the effects of stripping of forms on the temperature of concrete 3 in. from the vertical surface are presented. The solid lines represent temperatures 3 in. from the surface with the forms in place; the broken lines represent the temperatures after the forms have been removed. When the forms are removed at 3 days it is seen that the concrete has just reached its maximum temperature, and, at 3 in. from the form, the temperature of the concrete drops over 20 F. by the following day.

*Roy W. Carlson & L. R. Forbrich; Correlation of Methods for Measuring Heat of Hydration of Cement. *Ind. Engr. Chemistry* (Anal. Ed.) 10 (7), 382-6, 1938.

If the forms are allowed to remain in place for 5 days, the computed temperature drop following the removal of forms is only about 10 F. because the temperature of the concrete has been slowly falling between the 3rd and 5th days. However, allowing the forms to remain in place 5 days instead of 3 days does not affect the total temperature drop from the maximum, but it does reduce materially the rate of temperature drop, which, of course, is important because the slow rate allows plastic flow to relieve stress more effectively. It would seem, therefore, that removal of the forms is desirable as early as possible, before the maximum temperature is reached, and that it would be advantageous both to delay and reduce the maximum. If the forms are not to be removed before the maximum is reached then removal as late as possible would seem beneficial.

In the upper diagram of Fig. 2 the computed temperature distribution from the vertical formed surface to a point 6 ft. within the monolith is shown at the time the forms are stripped and 2 days later. The shaded portions of the diagram represent temperature drop following form removal. While cracking of this concrete, especially at the surface, may not occur, it probably is not far from the critical point for 4 sack concrete. As mentioned before, the magnitude of these effects, and hence the danger of crack formation would be dependent further upon the ambient temperature, particularly upon rapid temperature changes.

Temperature variations in concrete walls

In Fig. 3 the temperature variations in an 18-in. concrete wall are shown. The concrete was assumed to be made with 5½ sacks of standard (normal) portland cement and the forms to be ¾ in. plywood. Both sides of the wall were exposed to the air and the forms were stripped at the end of 2 days.

In the lower diagram of Fig. 3 the computed temperature rise of the concrete 3 in. from the formed surface is shown; the broken portion of the curves represent the temperature drop following form removal. The diagram shows that a temperature rise of more than 25 F. may occur at a point 3 in. from the surface up to the time the forms are removed. After the forms are removed a temperature drop of about the same magnitude may occur within two days, with almost the entire drop occurring during the first day following form removal.

In the upper diagram of Fig. 3, the computed temperature distribution in the same wall is shown. The curves represent only one half the thickness of the wall; the curves are symmetrical for the other half. It is clear that in a wall of this thickness made with 5½ sacks of cement per yard of concrete, temperature drops of 25 F. within 2 days, throughout the entire wall are possible. A temperature drop of this magnitude at

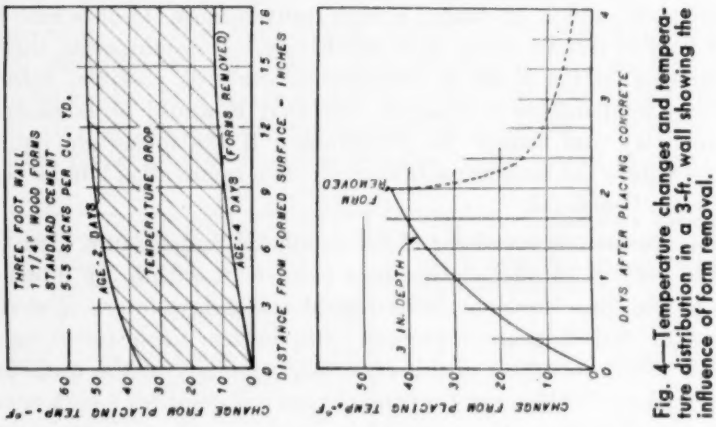


Fig. 4—Temperature changes and temperature distribution in a 3-ft. wall showing the influence of form removal.

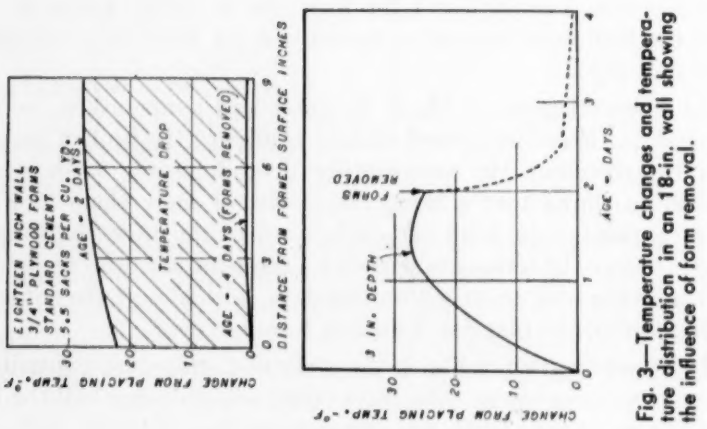


Fig. 3—Temperature changes and temperature distribution in an 18-in. wall showing the influence of form removal.

the age of 4 days would hardly be expected to cause cracking unless there were some structural weakness.

In Fig. 4 computed temperature variations in a 3 ft. wall illustrated in the same manner as those for the 18-in. wall are shown. In general the variations are similar except that the magnitude of temperature changes is greater for the 3-ft. wall. It is to be noted that a temperature drop of more than 40 F. may occur in this wall when the forms are removed at 2 days. A temperature drop of 40 F. for 5½ sack concrete might be expected to produce sufficient tensile stress to initiate cracking. If there are any weaknesses in the wall, especially at the surface, cracking to some degree at least, would probably result.*

In Fig. 5 computed temperature variations for a 6-ft. concrete wall are shown and in general the variations are about the same as for the 3-ft. wall except that the temperature changes are somewhat greater.

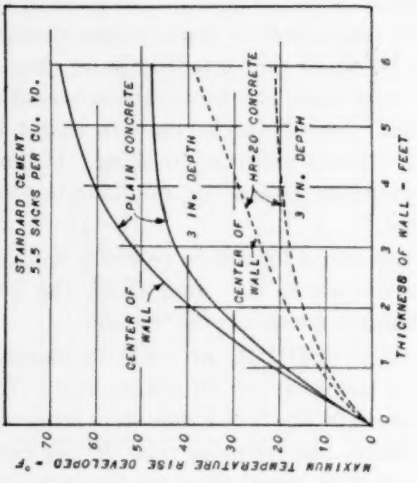
In Fig. 5 there is shown also the effect of HR-20, an additive material for reducing early heat liberation, on temperatures in a 6-ft. wall. The curves of the lower diagram show that up to the time forms were removed, (2 days) HR-20 reduced the temperature rise by more than 50 per cent. After the forms were removed the sudden rapid drop in temperature, shown by the curve for the plain concrete, was materially reduced by HR-20.

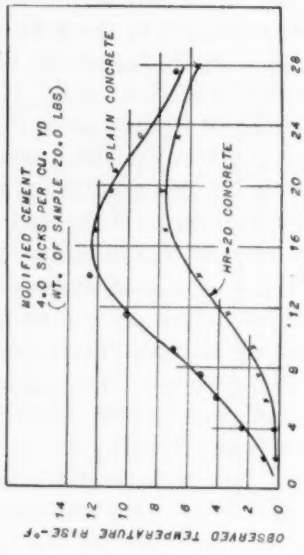
In the upper diagram of Fig. 5, the effect of HR-20 on temperature distribution is shown. Here again, HR-20 reduces markedly the temperature drop throughout the wall after the forms were stripped. Beyond about one foot from the surface a slight rise in temperature, rather than a drop in temperature, results. This is due to the delayed hydration of the cement, caused by the presence of HR-20.

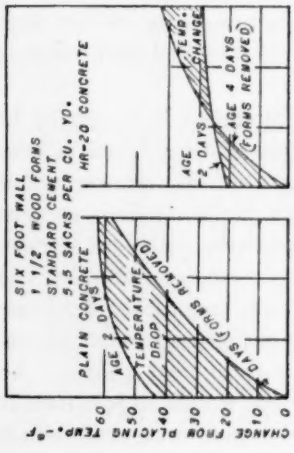
As mentioned previously, the effect of HR-20 shown here cannot be taken to represent accurately the effect that would be produced in concrete placed in the field. It should be recalled that the computations were based on heat of hydration measurements made on neat cement pastes cured at approximately constant temperatures. Recent experiments have shown that measurements made on neat cement pastes in the presence of HR-20 cannot be used to compute accurately temperature effects to be expected in concrete since the reduction in heat liberation due to HR-20 is not as great in concrete as in neat pastes. An approximation of what the actual effect in concrete would be is shown in Fig. 7. In this diagram actual temperature rise measurements, which were made on concretes placed in a simplified calorimeter, are presented.

In Fig. 6 the effects of wall thickness on the maximum temperature rise computed at a point 3 in. from the vertical formed surfaces and at

*Davis, Davis and Brown: Plastic Flow and Volume Changes of Concrete. Proceedings, Am. Soc. Testing Mats., Vol. 37, Part II, p. 328 (Thermal Stress Studies), 1937.







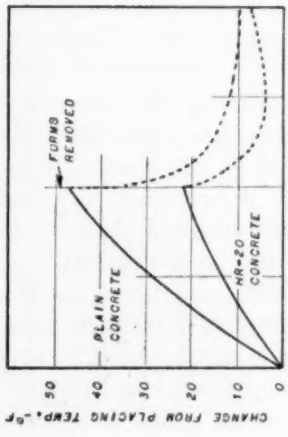


Fig. 5 (above)—Temperature changes and temperature distribution in a 6-ft. wall showing the effect of HR-20 and form removal.

Fig. 6 (top right)—Effect of wall thickness on the maximum temperature rise developed in concrete walls.

Fig. 7 (bottom right)—Effect of HR-20 on the observed temperature rise of concrete.

the centers of the walls are shown. The concrete was assumed to be the same as that considered in the walls previously, but with and without HR-20. The maximum temperatures developed represent the highest temperatures at the points in the wall indicated, up to the age of 4 days with the forms stripped at the end of 2 days. The relationships obviously would be different if the forms had been allowed to remain in place for a longer period; the difference, however, would be significant only for a wall thicker than 3 ft. In this diagram it is shown that the maximum temperature rise produced in a concrete wall is dependent to a considerable degree on wall thickness; the maximum rise increases with wall thickness. At a 3-in. depth from the formed surface there is little increase with wall thickness in the maximum temperature rise beyond about a 3-ft. thickness. At the center of the wall, however, the maximum rise is affected to a considerable degree by wall thickness even beyond 6 ft., the thickest wall considered. This diagram also suggests that in walls thicker than about two feet, temperature rise just below the formed surface is of such a magnitude that its effect cannot safely be disregarded. At the center of the wall higher maximum temperatures are developed as would be expected, but as pointed out previously, these probably have less significance except insofar as they affect surface temperatures.

The computations represented by the curves of Fig. 3, 4, 5, and 6 illustrate that cracking of concrete due to thermal effects may not be confined entirely to what is commonly termed "mass concrete." In the computations pertaining to the walls, conditions were imposed which are commonly used in modern practice. It is not difficult, therefore, to understand why relatively thin structures very frequently show unexpected and unaccountable cracking. It seems likely that cracking which has been attributed to drying shrinkage, or faulty construction, may actually be caused by temperature effects in many cases, or by the combined effects of temperature and shrinkage. It follows that attention should be paid to the time at which forms are stripped and to means of reducing maximum temperatures even in structures of moderate thickness.

SUMMARY

Calculations have been made to show the temperature effects at or near the surfaces under forms for typical concrete structures. These calculations have led to certain conclusions as follows:

1. Cracking because of temperature effects may occur in relatively thin sections. Maximum temperature rises near the surface from 25 to 45 F. may be expected in 1½- to 6-ft. sections under usual job conditions.

2. Removal of forms as early as possible before maximum temperatures are attained is indicated as a means of reducing the danger of cracking.

3. If removal of forms before the maximum is reached is impractical, then leaving the forms in place as long as possible should be beneficial.

4. Reduction of early heat liberation with a material such as HR-20 should be effective in reducing temperature rise and minimizing cracking from this cause.

Acknowledgements are due to Roy W. Carlson, Massachusetts Institute of Technology, and J. E. Deignan, U. S. Engineer Office, Pittsburgh, for valuable suggestions and help in making the computations involved. The writer is also indebted to E. W. Scripture, Jr., Master Builders Research Laboratories, for many valuable suggestions in preparing this paper.

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Tests of Reinforced Concrete Beams with Recommendations for Attaining Balanced Design*

By KENNETH C. COX†

SYNOPSIS

Upon the basis of test results from 110 rectangular beams of 4 concrete strengths and 23 percentages of reinforcement, recommendations are offered for attaining a balanced design. These are similar to Whitney's recommendations and embody a simplifying modification of the technique outlined by Whitney. These results include limited data on doubly-reinforced beams and beams of variable effective depth.

INTRODUCTION

A program was designed to determine experimentally the maximum amount of steel that can be fully utilized for any concrete strength. This question, practical in nature, should indicate the most economical design for flexure in reinforced concrete.

TEST PROGRAM AND PROCEDURE

The essential data on the concrete and the fabrication and testing of specimens should be apparent from Tables 1 to 4 and Fig. 1 to 4. The beams were loaded through springs to attain the equivalent of dead-weight loading. Deflection readings were made "on the run". Test results are tabulated in Tables 3 to 5, inclusive; the accompanying curves are self-explanatory.

DISCUSSION OF TEST RESULTS

General

The test results were, in general, as uniform as might be expected. This is particularly true of all the beam tests. For beams made in triplicate, the deviation of the results from the average value was negligible.

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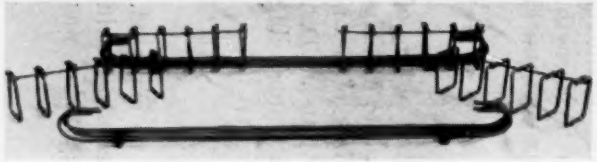
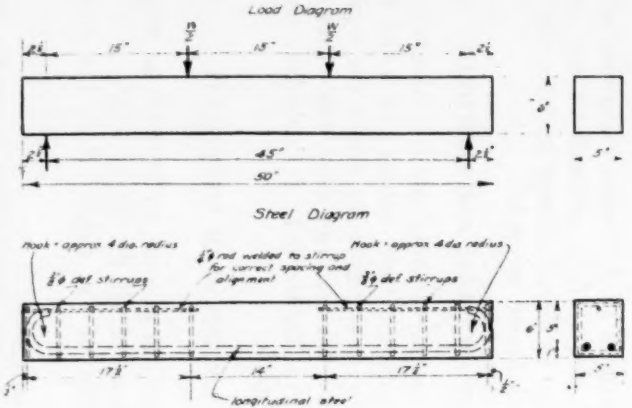
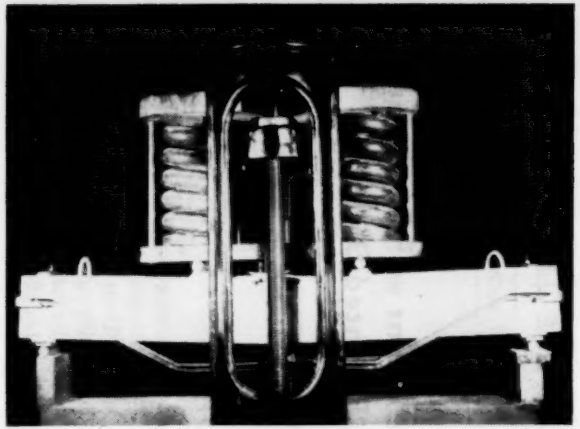
†Engineer, Contracting Division, Dravo Corp., Pittsburgh, Pa., formerly Concrete Reinforcing Steel Institute Research Fellow, Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pa.







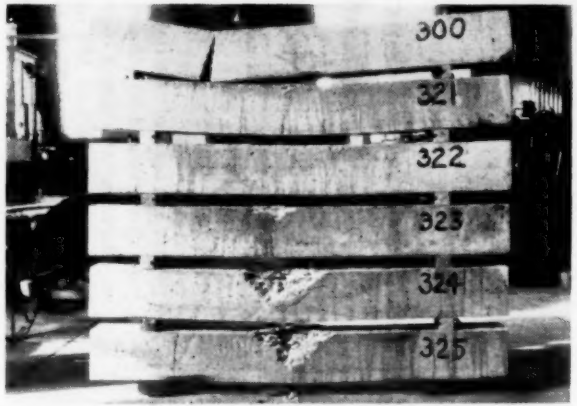


Fig. 1 (top left)—Typical reinforcing

Fig. 2 (bottom left)—Load and steel diagrams for the beams

Fig. 3 (top right)—Test set up.

Fig. 4 (bottom right)—Several beams after testing.

Beam tests and balanced design data

The data obtained from the beam tests show very clearly, in the case of the concrete with cement-water ratios of 1.00 and 1.25, just what percentage of steel results in a "balanced design".

The percentage of steel for balanced design can most easily be obtained by studying the curves in which the ultimate load is plotted against the percentage of steel. A break occurs in the curve at that percentage of steel required for balanced design. This point or break occurs when the steel yield-point strength and compression strength of the concrete are reached at the same time. Below this point the steel yields before the concrete fails; while above the break the concrete

TABLE 1—CONCRETE MATERIALS USED FOR THE VARIOUS CEMENT-WATER RATIOS

C/W Ratio by Weight	Cement lb.	Water* lb.	Sand lb.	Gravel lb.	Average Compressive Strength All Sizes p.s.i.	Number of Cylinders in Average
1.00	27.1	29.8	108	163	1970	9
1.25	33.8	29.8	106	160	3060	33†
1.50	40.6	29.7	104	153	4750	9
1.75	47.4	29.7	102	153	5870	9

Data on Compressive Strength of Cylinders

C/W Ratio by Weight	Number and Size of Cylinders (Vibrated)	28-Day Compressive Strength psi.	Number and Size of Cylinders (Rodded)	28-Day Compressive Strength psi.	Number and Size of Cylinders (Rodded)	28-Day Compressive Strength psi.
1.00	3, 6 x 12 in.	1810	3, 6 x 12 in.	1920	3, 3 x 6 in.	2170
1.25	3, 6 x 12 in.	3220	3, 6 x 12 in.	3340	3, 3 x 6 in.	3360
1.50	3, 6 x 12 in.	4650	3, 6 x 12 in.	4650	3, 3 x 6 in.	4950
1.75	3, 6 x 12 in.	5470	3, 6 x 12 in.	5690	3, 3 x 6 in.	6360

*One per cent of weight of aggregate added for absorption.
†24 additional vibrated cylinders included.
Note: Design based on 325 lb. of water per cu. yd. for workability required.

TABLE 2—PHYSICAL PROPERTIES OF THE STEEL

Bar Size in In. ϕ	Yield Load lb.	Yield Stress psi.	Ultimate Stress psi.	Per Cent Elongation		Per Cent Reduction in Area	Number of Specimens Tested
				2 in.	8 in.		
¼ def.	2,670	55,200	76,500	26.5	18.1	63.9	5
⅜ def.	5,880	53,400	82,500	26.5	19.1	50.9	10
½ def.	9,290	48,100	78,000	33.5	22.8	57.0	5
⅝ def.	14,690	48,100	78,600	31.5	22.3	45.5	5
¾ def.	21,970	50,600	77,700	39.0	25.6	48.4	3

TABLE 3—SUMMARY OF DATA ON BEAMS HAVING A CEMENT-WATER RATIO OF 1.25

Beam No.	Size (Inches) and Kind of Bars ϕ	Yield Strength of Steel lb.	Average Structural Limit Load lb.	Average Ultimate Load lb.	Deflection at Structural Load lb.	Deflection at Ultimate Load lb.	Per Cent Steel	Area of Steel Sq. In.	Stirrups in Each End of Beam
221	2, $\frac{3}{8}$ def.	11,760	6,480	7,490	0.135	0.852	0.88	0.22	6
222	4, $\frac{3}{8}$ def.	23,520	12,690	12,800	.179	.420	1.76	.44	6
223	6, $\frac{3}{8}$ def.	35,280	17,920	18,000	.229	.363	2.64	.66	6
224	8, $\frac{3}{8}$ def.	47,040	17,640	17,640	.233	.233	3.52	.88	8
225	10, $\frac{3}{8}$ def.	58,800	18,160	18,160	.225	.225	4.40	1.10	9
241	1, $\frac{5}{8}$ def.	14,690	8,700	8,810	.144	.545	1.20	.30	6
242	2, $\frac{5}{8}$ def.	29,320	16,090	16,100	.191	.306	2.44	.61	6
243	3, $\frac{5}{8}$ def.	44,070	19,020	19,020	.226	.226	3.68	.92	8
244	4, $\frac{5}{8}$ def.	58,760	19,300	19,300	.198	.198	4.88	1.22	9
211	4, $\frac{1}{4}$ def.	10,680	6,980	7,150	.131	.373	0.76	.19	6
212	8, $\frac{1}{4}$ def.	21,360	12,970	13,180	.176	.388	1.56	.39	6
213	12, $\frac{1}{4}$ def.	32,040	17,900	17,010	.223	.265	2.32	.58	6
214	16, $\frac{1}{4}$ def.	42,720	19,240	19,240	.238	.238	3.08	.77	8
215	20, $\frac{1}{4}$ def.	53,400	19,150	19,150	.255	.255	3.88	.97	8
231	1, $\frac{1}{2}$ def.	9,290	5,540	6,480	.120	1.250	0.76	.19	6
232	2, $\frac{1}{2}$ def.	18,580	11,030	11,340	.166	.450	1.56	.39	6
233	3, $\frac{1}{2}$ def.	27,870	15,410	15,730	.183	.331	2.32	.58	6
234	4, $\frac{1}{2}$ def.	37,160	17,910	17,910	.320	.320	3.08	.77	8
235	5, $\frac{1}{2}$ def.	46,450	19,580	19,580	.243	.243	3.88	.97	8
251	1, $\frac{3}{4}$ def.	21,970	11,900	12,300	.181	.357	1.72	.43	6
252	2, $\frac{3}{4}$ def.	43,940	19,490	19,490	.237	.237	3.48	.87	8
253	3, $\frac{3}{4}$ def.	65,910	19,980	19,980	.202	.202	5.20	1.30	9
200	0 bars	0	1,830	1,830	.010	.010	0	0	0

c/w = 1.25

All above figures are the average of three specimens

TABLE 4—SUMMARY OF DATA ON BEAMS HAVING CEMENT-WATER RATIOS OF 1.00 AND 1.50

Beam No.	Size (Inches) and Kind of Bars ϕ	Yield Strength of Steel lb.	Average Structural Limit Load lb.	Average Ultimate Load lb.	Deflection at Structural Load lb.	Deflection at Ultimate Load lb.	Per Cent Steel	Area of Steel Sq. In.	Stirrups in Each End of Beam
100	0 bars	0	1,310	1,310	0	0.005	0	0	0
121	2, ⅜ def.	11,760	6,450	6,650	.158	.295	0.88	0.22	6
122	4, ⅜ def.	23,520	11,230	11,380	.233	.312	1.76	.44	6
123	6, ⅜ def.	35,280	11,910	11,910	.218	.302	2.64	.66	6
124	8, ⅜ def.	47,040	12,800	12,800	.201	.235	3.52	.88	6
125	10, ⅜ def.	58,800	13,250	13,250	.200	.260	4.40	1.10	6
141	1, ⅝ def.	14,690	7,840	8,070	.150	.270	1.20	.30	6
142	2, ⅝ def.	29,380	12,440	12,440	.242	.320	2.44	.61	6
143	3, ⅝ def.	44,070	11,980	11,980	.195	.295	3.68	.92	6
144	4, ⅝ def.	58,760	13,800	13,800	.184	.244	4.88	1.22	6
				c/w = 1.00					
300	0 bars	0	1,200	1,200	0	0.004	0	0	0
321	2, ⅜ def.	11,760	7,350	8,520	.118	1.250	0.88	0.22	6
322	4, ⅜ def.	23,520	13,760	14,170	.157	.468	1.76	.44	6
323	6, ⅜ def.	35,280	19,300	19,600	.200	.395	2.64	.66	6
324	8, ⅜ def.	47,040	22,000	22,000	.258	.258	3.52	.88	8
325	10, ⅜ def.	58,800	23,920	23,920	.230	.230	4.40	1.10	9
341	1, ⅝ def.	14,690	9,320	10,210	.148	.710	1.20	.30	6
342	2, ⅝ def.	29,380	16,030	16,610	.172	.322	2.44	.61	6
343	3, ⅝ def.	44,070	22,200	22,200	.210	.210	3.68	.92	8
344	4, ⅝ def.	58,760	25,580	25,580	.224	.224	4.88	1.22	9
				c/w = 1.50					

All the above figures are of a single test specimen.

TABLE 5—SUMMARY OF DATA ON BEAMS HAVING CEMENT-WATER RATIOS OF 1.25 AND 1.75

(Compressive Steel and Various Depth Beams)

Beam No.	Size (Inches) and Kind of Bars ϕ	Yield Strength of Steel lb.	Average Structural Limit Load lb.	Average Ultimate Load lb.	Deflection at Structural Load lb.	Deflection at Ultimate Load lb.	Per Cent Steel	Area of Steel Sq. In.	Stirrups in Each End of Beam
400	0 bars	0	2,550	2,550	0.009	0.009	0	0	0
421	2, ⅜ def.	11,760	7,280	8,950	.120	1.250	0.88	.22	6
422	4, ⅜ def.	23,520	13,480	14,020	.149	.418	1.76	.44	6
423	6, ⅜ def.	35,280	19,980	20,540	.186	.342	2.64	.66	6
424	8, ⅜ def.	47,040	23,980	24,630	.231	.292	3.52	.88	8
425	10, ⅜ def.	58,800	28,270	28,270	.265	.265	4.40	1.10	9
441	1, ⅝ def.	14,690	8,900	10,550	.118	1.050	1.20	.30	6
442	2, ⅝ def.	29,380	16,570	17,570	.186	.438	2.44	.61	6
443	3, ⅝ def.	44,070	24,080	24,690	.203	.350	3.68	.92	8
444	4, ⅝ def.	58,760	28,960	28,960 c/w = 1.75	.195	.245	4.88	1.22	9
				Varying Depth Beams					
222x5	4, ⅜ def.	23,520	9,820	10,120	.250	.380	2.20	.44	8
222x7	4, ⅜ def.	23,520	16,770	16,770	.220	.480	1.47	.44	8
222x8	4, ⅜ def.	23,520	20,100	21,130	.130	.300	1.26	.44	8
222x9	4, ⅜ def.	23,520	22,800	24,280	.164	.594	1.10	.44	8
222x10	4, ⅜ def.	23,520	26,500	28,000	.106	.600	0.98	.44	8
222x11	4, ⅜ def.	23,520	29,130	31,000	.105	.600	0.88	.44	8
			Beams with Compression Reinforcing						
244-C1	4, ⅝ def.	58,760	22,670	22,670	.238	.332	4.88*	1.22*	13
244-C2	4, ⅝ def.	58,760	25,260	25,260	.254	.348	4.88*	1.22*	13
244-C4	4, ⅝ def.	58,760	29,320	29,320 c/w = 1.25	.252	.360	4.88*	1.22*	13

*Does not include compression steel. Compression steel used for Beams 244-C1, 244-C2, and 244-C4 was 1, 2, and 4, ¾-in. diameter bars respectively, placed 1¼ in. from the top face.

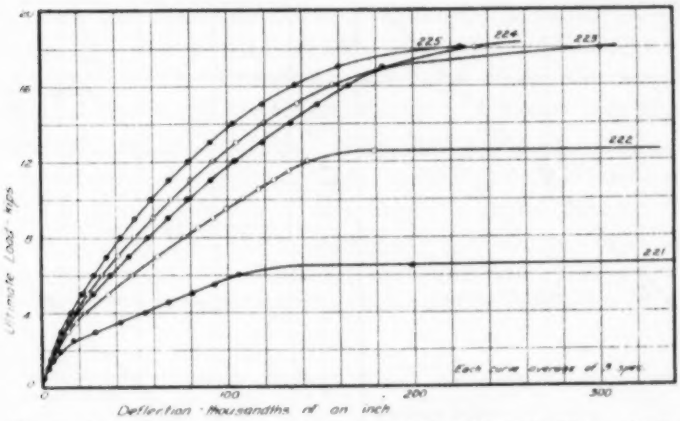


Fig. 5—Load-deflection curves for beams 221, 222, 223, 224 and 225

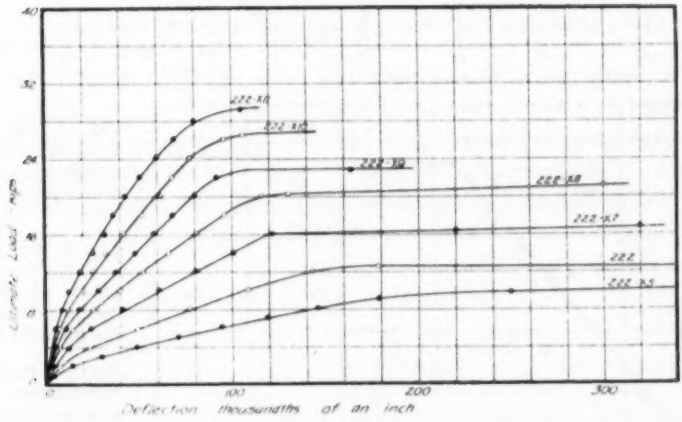


Fig. 6—Load-deflection curves for beams 222-X5, 222, 222-X7, 222-X8, 222-X9, 222-X10, and 222-X11

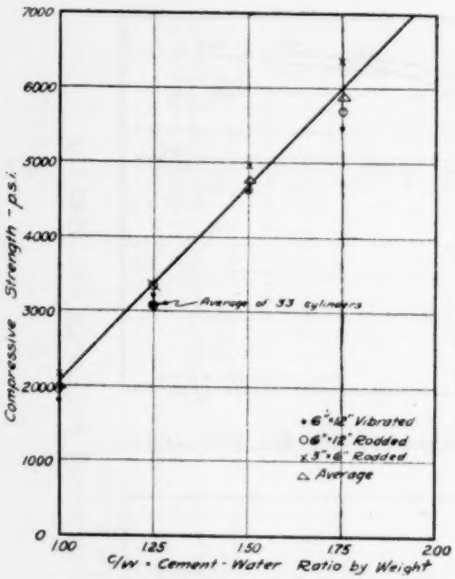
crushes before the steel reaches its yield point. Hence the breaking point is the measure of the percentage of steel for balanced design.

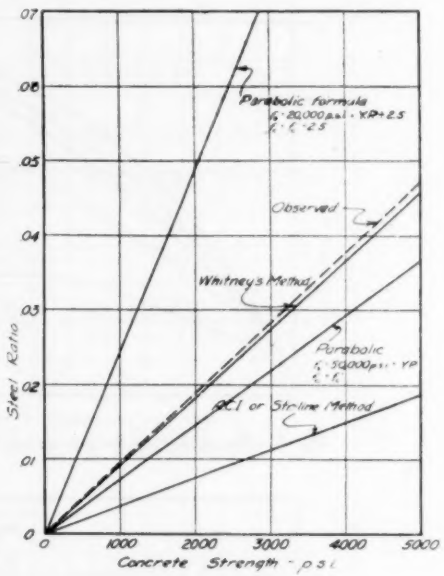
Examining the percentages of steel required to produce balanced design and substituting these percentages in the formula

$$p_m = C' \frac{f_c}{f_s}$$

with the correct values of f_c and f_s we will find that C' is approximately 0.47.

In the case of the high cement-water ratios, or higher strength concretes, the maximum percentage of steel for balanced design is not such a distinct point as we have in the lower cement-water ratio concretes.





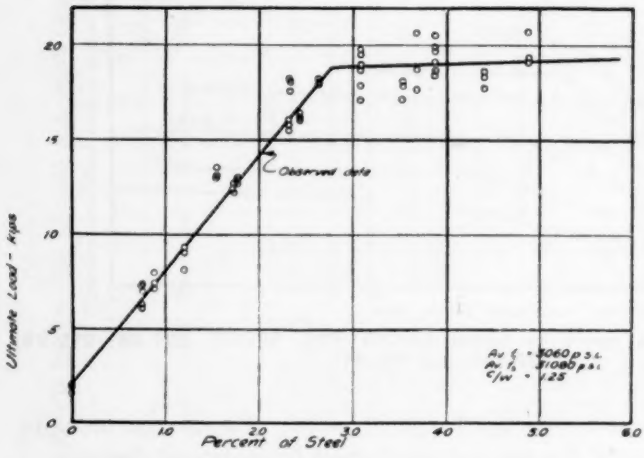


Fig. 7 (top left)—Relation between compressive strength and cement - water ratio

Fig. 8 (bottom left)—Curve showing the effect of the percentage of steel on the ultimate load

Fig. 9 (above)—A comparison of the steel ratios allowed by various design methods for different concrete strengths

If an estimate is made based on the data obtained for the maximum percentage of steel with concrete having a cement-water ratio by weight of 1.00 or 1.25, it can be seen that there was not enough steel placed in the beams of cement-water ratios 1.50 and 1.75 to reach a balanced design. Perhaps in many cases such a high percentage of steel would be impracticable because of the difficulty which might be encountered in arranging the steel and placing the concrete.

Fig. 9 compares the maximum steel ratios allowed by the straight line method, the parabolic method, Whitney's[(1)]* method, and an equation

*See (1) of Bibliography.

derived from the observed data. Whitney's method, an empirical method, has been based on actual tests performed by Slater and Lyse at Lehigh University[2], by Humphrey and Losse of the Bureau of Standards[1], and by the U. S. Geological Survey. Perhaps this is why the observed results are in line with Whitney's conclusions.

A proposed method suggesting slight variations to Whitney's proposals derived from material given in the tables is found below. The first assumption made is that of rectangular distribution of stress, i.e., there is a rectangular block which has an equal distribution of stress over a cross section of the concrete near the top of the beam.

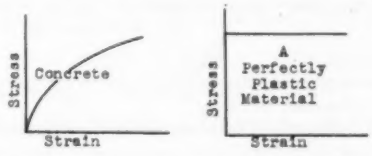


Fig 10—Comparison of stress strain curves of concrete and a perfectly plastic material

This assumption is made because concrete behaves to some extent as a plastic material, and the assumption of the stress-strain diagram of a perfectly plastic material gives results which are in close agreement with the breaking strength of beams. (See Nadii's *Plasticity*). Upon this assumption and with the percentage of steel for balanced desgn obtained above, the proposed method is based.

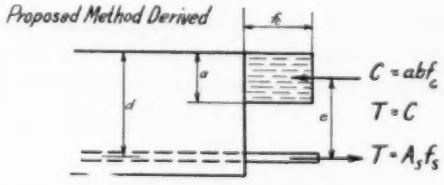


Fig. 11—Rectangular distribution

$$M_s = A_s c f_s = M_c = abf_c c$$

$$A_s f_s = abf_c$$

$$\frac{a}{2} = \frac{A_s f_s}{2bf_c}$$

$$M = \left(d - \frac{A_s f_s}{2bf_c}\right) A_s f_s = \left(d - \frac{A_s f_s}{2bf_c}\right) abf_c = \left(d - \frac{pf_s d}{2f_c}\right) pbdf_s$$

$$\frac{M}{bd^2} = pf_s\left(1 - \frac{pf_s}{2f_c}\right) \quad \ldots\ldots\ldots\

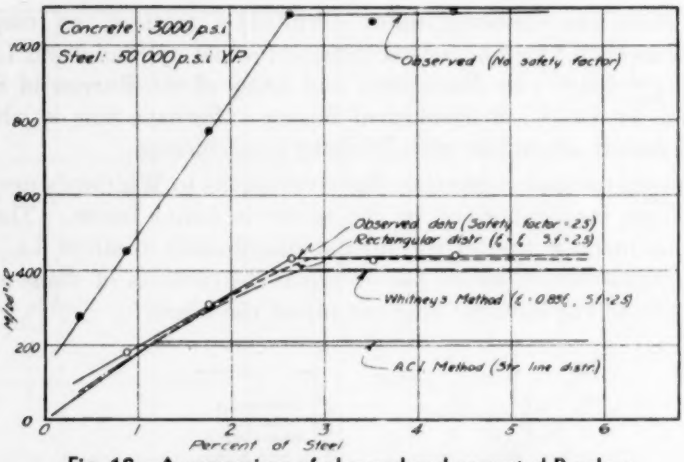


Fig. 12—A comparison of observed and computed R values

data agree very closely. Working stresses of forty per cent of the yield-point strength of the steel and forty per cent of the compressive strength of the concrete were used in each method. The observed data were plotted with no safety factor and also with forty per cent of the observed $\frac{M}{bd^2}$ values.

The straight-line method of design needs no explanation, since it is the prevailing method. Whitney's method assumes a rectangular stress distribution in the compression face of a rectangular concrete beam. He sets the design strength of the concrete to be 0.85 of the cylinder strength. The balanced design percentage of steel given by Whitney is:

$$p_{max} = 0.456 \frac{f_c}{f_s} \quad \ldots\ldots\ldots\

The conventional straight-line method will not be given in detail in this paper. (See Turneaure and Maurer, reference (3) for derivation.) The derivation of the formula for the percentage of steel for a balanced design using the straight line distribution of stress results in the formula:

$$p_{max} = \frac{f_s}{f_c}\left(\frac{f_s}{2nf_c} + 1\right)^{1/9} \quad \ldots\ldots\ldots\

By setting a definite safety factor in the proposed method or in Whitney's method there is an "overall" factor for the beam as a structural unit. The straight line method results in a high factor of safety for the concrete and the desired factor in the steel.

The tests made on beams with compression reinforcing and the resulting computations show that the proposed theory is also adaptable to beams reinforced with compression steel. Fig. 13 illustrates how closely the additional strength due to the compression steel can be approximated. The method requires the determination of the strength of a beam without compression steel and then adding the strength contributed by the compression steel. The tensile steel area must equal the area required for balanced design with no compression steel plus an area equal to the area of compression steel used.

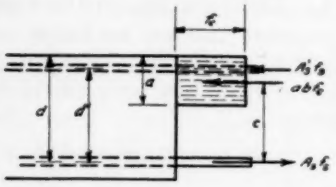


Fig. 14—Compression steel — a proposed method

The following expression has been derived previously (see Equation 1).

$$\frac{M}{bd^2} = pf_s\left(1 - \frac{pf_s}{2f_c}\right)$$

For balanced design, $p = 0.47\frac{f_c}{f_s}$ therefore $\left(1 - \frac{pf_s}{2f_c}\right) = 0.77$

$$M = pf_s bd^2\,(0.77) + A_s'f_s d'$$

The total steel in the compression side will be A_s'. The total steel area in the tensile side will be the percentage required for balanced design plus an amount of steel equal to the compression steel, or:

Total area of tensile steel $= pbd + A_s'$

The proposed method is satisfactory for varying depths of beam as shown by Fig. 15 which compares the computed and observed data for various beam depths. The observed data show that the ultimate loads lie slightly above the computed curve for the deepest beams and very close to the computed curve for shallow beams. This can be explained by the fact that the "reserve strength" between the structural yield of a beam and the ultimate load of the beam are closer together for high percentages of steel than for low percentages of steel. One curve was drawn using the structural yield load in place of the ultimate load; this resulted in better correlation with the computed curve.

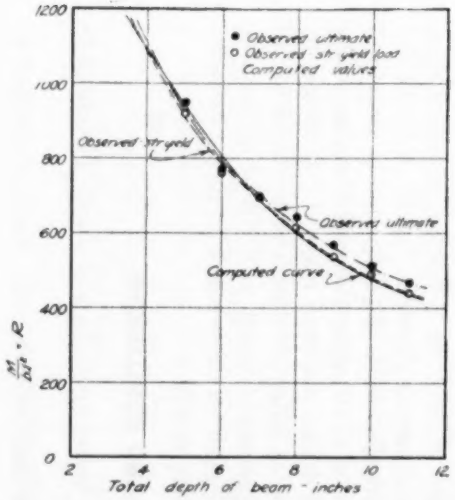


Fig. 15 (above)—A comparison of observed and computed R values for beams of various depth

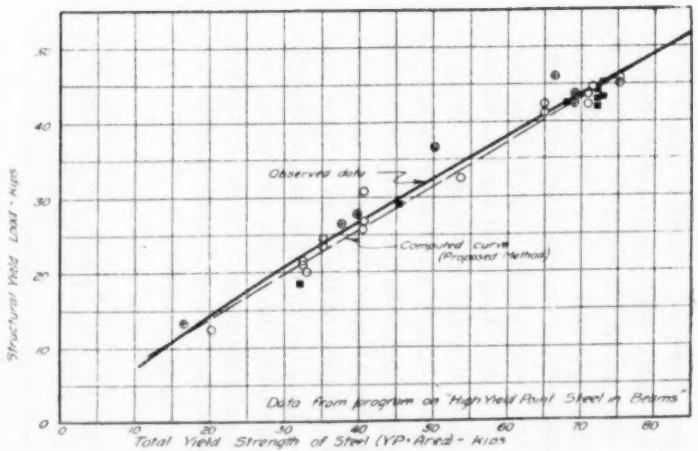

Observed data
Computed curve (Proposed Method)
Data from program on "High Yield Point Steel in Beams"
Total Yield Strength of Steel (YP·Area) - kips

Fig. 16 (top right)—Relation between structural yield and total yield strength of reinforcing steel

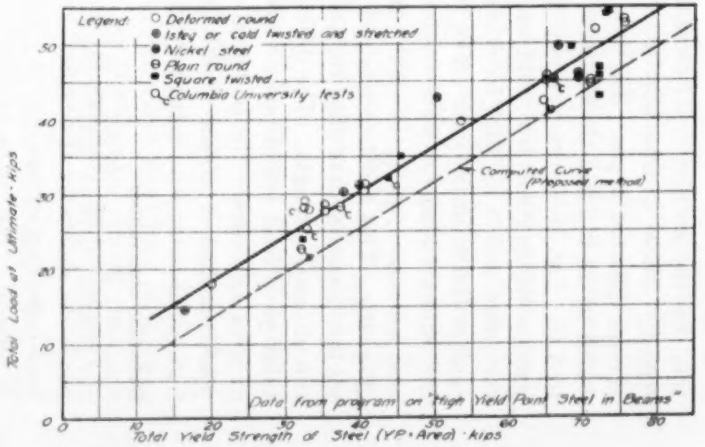

Legend:
Deformed round
Isteg or cold twisted and stretched
Nickel steel
Plain round
Square twisted
Columbia University tests
Computed Curve (proposed method)
Data from program on "High Yield Point Steel in Beams"
Total Yield Strength of Steel (YP·Area) - kips

Fig. 17 (bottom right)—Relation between ultimate strength of beams and total yield strength of reinforcing steel

Comparison with other tests

In a recent investigation carried on at the Fritz Engineering Laboratory by the author[14], 32 rectangular reinforced concrete beams were tested. These beams were reinforced with high yield-point steel of several different types. The beams had an effective depth of 12 in., a width of 12 in., and the distance center-to-center of supports was equal to 9 ft. Fig. 16 shows how closely the proposed method of design compares with the structural yield load of these 32 beams. Fig. 17 illustrates the same points as Fig. 16 except that the computed load is plotted against the ultimate load. The reader will notice a reserve strength in the ultimate loads over the computed values in Fig. 17 which is not the case when comparing the structural yield load with the computed values (Fig. 16). This can be explained by a point mentioned previously, i.e., for low percentages of steel the structural yield of the beam is lower than the ultimate strength. The difference between ultimate load and structural yield load (yield point x area of steel) diminishes as the per cent of steel approaches the balanced design percentage. In short, the proposed method of design would apply to the 32 rectangular beams with high yield-point steel using any desired factor of safety.

Deflection observations

The deflection readings taken "on the run" gave very consistent results as shown by the typical curves of Fig. 5. The first break in the load-deflection curves near the low loads indicates failure of the concrete in tension. The curve advances approximately in a straight line to the next break, marking the initial yielding of the steel which eventually causes failure.

In beams with low percentages of steel the deflection at any load is far greater than in the high percentages as shown in Fig. 5. It is evident from an examination of this figure that when the deflection at structural yielding of the beams and deflection at the ultimate reach the same value the critical point for a balanced design has been reached. It should be noted that the average deflection at balanced design percentages for all strengths of concrete was approximately 0.200 to 0.210 of an inch or 0.04 to 0.05 per cent of the span length for the beams with no compression reinforcing and five inches effective depth.

SUMMARY AND CONCLUSIONS

The following conclusions are based upon the test results presented in this paper:

1. For balanced design the maximum steel ratio varies according to the formula $p_{max} = 0.47 \frac{f_c}{f_s}$.

2. Whitney's method ($f_c' = 0.85f_c$ with rectangular distribution) more closely approximates actual test results than the parabolic or straight-line distribution methods. Whitney's method and the proposed method give equally good results. The proposed method falling closer to the observed data in the concrete strengths around 3000 psi.

3. The proposed method is in closer agreement with actual data than the common straight-line distribution method.

4. The proposed method has the advantage of being the most simple to apply and the easiest to understand of all the design methods studied.

5. The results of 110 beams tested in this paper and 32 large beams in a program carried on by the author indicate that the proposed method is adaptable to every size and depth of beam.

6. At balanced design the straight-line theory with its present limitations uses only 40 per cent as much steel as could be used with the present factors of safety.

7. The yield point of compression steel is reached before buckling takes place in the compression steel.

8. The proposed method includes the use of compression steel, affording simplicity of design.

9. The reserve strength between the structural yield load of a beam and the ultimate strength is greater at low percentages of steel than it is at high percentages, diminishing as the percentage of steel approaches a balanced design.

10. The test results show that additional steel over and above balanced design does not add material strength to a beam.

11. Additional steel over that required for a balanced design will reduce the deflection for any given load and may in many cases be an important consideration in design.

NOMENCLATURE

p_m = steel ratio for balanced design
C' = a constant
f_c = cylinder strength of concrete, pounds per square inch
f_s = yield-point strength of steel, pounds per square inch
d = effective depth of beam, inches
a = depth of concrete assumed to contribute full compression strength
b = breadth of rectangular beam
c = lever arm of steel reinforcement
C = total compression in concrete
T = total tension in steel
M = bending moment or resisting moment, in general
M_c = resisting moment of concrete
M_s = resisting moment of steel
A_s = area of tensile steel
A_s' = area of compression steel
d' = lever arm between tensile and compression steel

ACKNOWLEDGMENT

The test specimens were fabricated and the tests were made in the Fritz Engineering Laboratory at Lehigh University. Acknowledgment

is made to the Concrete Reinforcing Steel Institute and to the Carnegie-Illinois Steel Corp. for furnishing the reinforcing steel; to the Lehigh Portland Cement Co. for the cement; to the Fritz Engineering Laboratory for the remaining materials; and to Inge Lyse, formerly Research Professor of Engineering Materials and now Professor of Reinforced Concrete and Solid Bridges at the Norges Tekniske Hiskole at Trondheim, Norway; Bruce Johnston, Assistant Director, Fritz Engineering Laboratory; Howard Godfrey, Engineer of Tests at Fritz Engineering Laboratory; and other members of the Fritz Engineering Research Staff for their assistance in preparation and testing of the specimens as well as for their continued interest and suggestions.

BIBLIOGRAPHY

(1) C. S. Whitney: Design of Reinforced Concrete Members under Flexure or Combined Flexure and Direct Compression. ACI JOURNAL, March-April 1937; Proceedings V. 33.

(2) W. A. Slater and Inge Lyse: Compressive Strength of Concrete in Flexure as Determined from Tests of Reinforced Beams. ACI JOURNAL, June 1930; Proceedings V. 26, p. 831.

(3) Turneaure and Maurer: Principles of Reinforced Concrete Construction.

(4) Inge Lyse: A Study of the Quality, the Design, and the Economy of Concrete. Franklin Institute JOURNAL, 1936, V. 221, p. 495, 653 and 745; V. 222, p. 83.

(5) M. O. Withey: Tests on Plain and Reinforced Concrete. Series of 1907, Bulletin 197, University of Wisconsin Engineering Series.

(6) Dean Peabody, Jr.: The Design of Reinforced Concrete Structures.

(7) K. Hajnal-Konyi: The Modular Ratio: A New Method of Design Omitting *m*. *Concrete & Constructional Engineering*, Jan. 1937.

(8) Slater, Lord and Zipprodt: Shear Tests of Reinforced Concrete Beams. Technologic Paper, National Bureau of Standards No. 314.

(9) A. N. Talbot: A Test of Three Large Reinforced Concrete Beams. University of Illinois Bulletins 28 and 29.

(10) Richart and Larson: An Investigation of Web Stresses in Reinforced Concrete Beams. University of Illinois Bulletins 175 and 176.

(11) Walker: Modulus of Elasticity of Concrete. Bulletin 5, Structural Materials Research Laboratory, Lewis Institute, Chicago, Ill.

(12) A. N. Talbot: University of Illinois Bulletins 1, 4, 10, 14.

(13) VanLougendenck: Os Novos Methodos de Dimensionamento Das Pecas Flectidas de Concerto Armado. Seperato de Boletim de Institute de Eugenharis de Junho de 1937.

(14) Johnston and Cox: High Yield Point Steel as Tension Reinforcement in Beams. ACI JOURNAL, Sept.-Oct. 1939; Proceedings V. 36.

Discussion of this paper should reach the ACI Secretary in triplicate by Mar. 1, 1942, for publication in the JOURNAL for June, 1942

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Vol. 13 No. 6 7400 SECOND BOULEVARD, DETROIT, MICHIGAN June 1942

Discussion of a paper by Kenneth C. Cox:

Tests of Reinforced Concrete Beams with Recommendations for Attaining Balanced Design*

DISCUSSION BY CHARLES S. WHITNEY and
PAUL W. ABELES

BY CHARLES S. WHITNEY†

Mr. Cox reports the results of an elaborate series of tests on rectangular beams which are, according to the writer's interpretation, entirely consistent with previous tests and with the formulas which have been proposed by the writer.[1][2] The author proposes not a new method but a modification of the empirical constants in the writer's formulas in the following manner:

(1) That the value of f_c be changed from $0.85\,f'_c$ to f'_c. It is assumed as before that f_c is the average ultimate compressive stress in the concrete represented by the equivalent rectangular stress block and f'_c is the standard cylinder strength (see page 487 reference [1]).

(2) That the steel ratio for balanced design be $0.47\frac{f'_c}{f_s}$ instead of $0.456\frac{f'_c}{f_s}$. Together with the change of f_c from $0.85f'_c$ to f'_c (which increases the steel lever arm) this has the effect of raising the value of M/bd^2 for a fully reinforced beam from $0.333f'_c$ to $0.36f'_c$.

Neither of these changes appears to be supported by the test data reported by Mr. Cox.

The first modification was suggested by Professor Morris, Professor Lyse, and Mr. Wernisch. The writer has previously stated that he preferred to place f_c equal to $0.85f'_c$ because it appears to give better

*ACI JOURNAL, Sept. 1941; *Proceedings*, V. 38, p. 65.

†Consulting Engineer, Milwaukee, Wisc.

[1] "Design of Reinforced Concrete Members under Flexure or Combined Flexure and Direct Compression" by Carles S. Whitney, ACI JOURNAL, March-Apr. 1937; *Proceedings*, V. 33, p. 483.

[2] "Plastic Theory of Reinforced Concrete Design" by Charles S. Whitney, *Proceedings*, Am. Soc. of Civil Engineers, Dec. 1940, p. 1749.

agreement with test results and because it is consistent with the strength shown by axially loaded columns (p. 498-28 ref. [1]). The latter is an important consideration because the writer's method applies to members loaded under the full range of eccentricity from zero (axial load) to infinity (pure bending).

The second modification gives a value for the ultimate moment of a beam with balanced reinforcement which is too high except for concretes of lower strength than those normally used. (Fig. 3, p. 490 ref. [1]). The writer's interpretation of the tests reported by Mr. Cox differs from the author's for two principal reasons.

In order to apply the formulas to a particular beam with less steel than the amount required for balanced reinforcement, it is necessary to know the strength of the steel in that particular beam. In these tests the yield stresses of the rods of various sizes varied from 48,100 to 55,200 psi. For that reason the comparison of tests with theory on the basis of average yield stresses as in Fig. 8, p. 72, is not accurate.

Furthermore, the term f'_c in the equations represents the strength of standard cylinders and not that of vibrated cylinders nor 3 in. by 6 in. cylinders. The author has evidently averaged the strengths of all three types instead of using only rodded cylinders. This resulted in a concrete cylinder strength of 3060 psi instead of 3340 psi for the series from which the author derived his $M/bd^2 = 0.36f'_c$ value. If he had used the rodded cylinders only the value of $M/bd^2f'_c$ would have been lower.

According to the writer's calculation the beams listed in Table A had more than enough steel for balanced reinforcement and the values of $M/bd^2f'_c$ are as tabulated. The value of p_o, the steel ratio for balanced reinforcement is $p_o = 0.456\frac{f'_c}{f_s}$. For the 3340 lb. concrete series the average value of $M/bd^2f'_c$ is $0.343 \pm .023$. For the Slater and Lyse tests (Table 1, page 488, ref. [1]) the average as shown in Table B is $0.335 \pm .019$. The result of each series is within the limits of error of the other. In the Slater and Lyse tests, cylinders were tested for each beam while in those reported by Mr. Cox only three rodded cylinders were tested for each series. Vibrated cylinders should not be used even though the beams are vibrated.

Along with the tests previously used by the writer (p. 498-30 ref. [1]) those reported by Mr. Cox have been plotted on Fig. A using the correct values for f_s and f'_c. The solid lines show the theoretical value of $M/bd^2f'_c$ based on the writer's recommendation and the dotted line

TABLE A—TESTS ON OVER-REINFORCED BEAMS REPORTED BY COX

Beam No.	$\frac{P}{P_o}$	f'_c	$\frac{M}{bd_2f'_c}$
224	1.235	3340	.317
225	1.543	3340	.326
243	1.160	3340	.343
244	1.540	3340	.346
214	1.115	3340	.346
215	1.405	3340	.345
235	1.223	3340	.352
252	1.156	3340	.350
253	1.728	3340	.360
Mean			.343 ± .023
122	1.072	1920	.356
123	1.610	1920	.373
124	2.140	1920	.402
125	2.680	1920	.413
142	1.340	1920	.389
143	2.020	1920	.373
144	2.680	1920	.432
Mean			.391
325	1.108	4650	.308
344	1.105	4650	.330
Mean			.319

TABLE B—TESTS ON OVER-REINFORCED BEAMS REPORTED BY SLATER AND LYSE

Beam No.	P	f'_c	$\frac{M}{bd^2f'_c}$
2	.028	2790	.337
3	.037	4070	.325
4	.047	4800	.339
5	.056	5740	.320
6A	.039	4130	.326
7	.028	2950	.341
10	.030	2820	.340
10A	.040	3810	.350
Mean			.335 ± .019

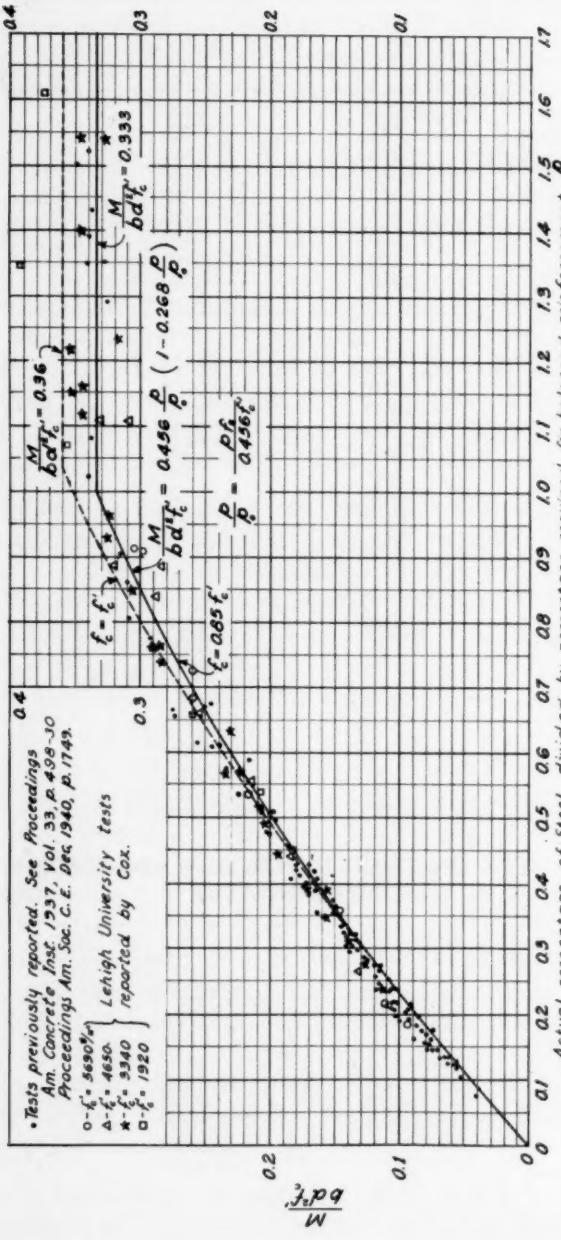
• Tests previously reported. See Proceedings Am. Concrete Inst. 1937, Vol. 33, p. 498-510 Proceedings Am. Soc. C.E. Dec. 1940, p. 1749.
○-f'c = 5690#/in²
△-f'c = 4650
★-f'c = 3340
□-f'c = 1920
Lehigh University tests reported by Cox.
fc = f'c
fc = 0.85 f'c
M/bd²f'c = 0.456 p/ρ (1 - 0.268 p/ρ)
M/bd²f'c = 0.36
M/bd²f'c = 0.333
p/ρ = ρfs/0.456f'c
M/bd²f'c
Actual percentage of Steel divided by percentage required for balanced reinforcement, p/ρ

Fig. A—Theoretical ultimate strength of concrete beams, compared with results of tests

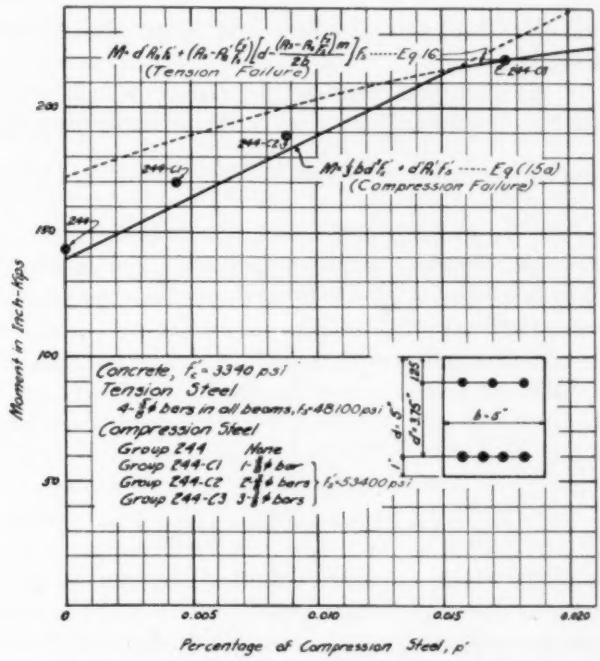

Fig. B—Effect of compressive steel on beam strength

gives the value based on the assumption that $f_c = f'_c$ and that the maximum $\frac{M}{bd^2f'_c}$ equals 0.36 as suggested by Mr. Cox.

It does not appear that the values represented by the dotted lines are justified by the tests. However the difference is not very great and it is gratifying that Mr. Cox, as the result of so extensive an investigation, has arrived at conclusions so close to the writer's.

The tests on beams with compression steel also show complete agreement with the writer's equations (15a) and (16)[2] as shown in Fig. B. According to these equations, Beam 244-C3 failed in tension and not in compression as assumed by Mr. Cox. He has evidently not considered the fact that the yield strength of the compression steel is 53,400 psi, while that of the tension steel is only 48,100 psi. The equivalent percentage of compression steel is $\frac{53{,}400}{48{,}100} \times 0.0176 = 0.0195$ instead of 0.0176. According to the writer's equation, $p_o = 0.456 \frac{3340}{48{,}100} + 0.0176 \frac{53{,}400}{48{,}100} = 0.0512$. The actual percentage of tension steel is 0.0488 and

this indicates why the beam did not develop its full strength in the compression side.

The area of tension steel required for balanced reinforcement in a beam subjected to flexure only is equal to the area required for balanced design with no compression steel plus $\frac{f'_s}{f_s}$ times the area of the compression steel. (f'_s is the yield strength of the compression steel.)

As shown by Fig. B, Beams 244-C1 and 244-C2 were over-reinforced in tension and developed slightly more than their theoretical strength in compression as given by Eq. (15a).

In the case of these beams with $p = 0.0488$, the amount of compression steel for balanced reinforcement is equal to

$$p'_s = \frac{f_s}{f'_s}\left(p - 0.456\frac{f'_c}{f_s}\right)$$

$$= \frac{48{,}100}{53{,}400}\left(0.0488 - 0.456\frac{3340}{48{,}100}\right) = 0.0154$$

This value corresponds to the intersection of the two curves of Fig. B. It is interesting to note that Beam 244-C3 which had the equivalent of about two per cent of compression steel (0.0195) and almost five per cent of tension steel (0.0488), failed at a load almost exactly equal to that given by Eq. (15a).

BY PAUL W. ABELES*

The use of the formula derived by Mr. Cox would result in a good agreement between design and actual behavior of reinforced concrete members of a balanced design, thus making full use of both the steel and concrete strength. Such a solution should be considered a great improvement; but from a practical and economical point of view many structures are designed to require less reinforcement than for balanced design. Such constructions with light reinforcement have become more common owing to the use of medium and high strength steel. In these cases the stresses at failure are in excess of the yield point stress as is known from various test results. This excess may be up to 50 per cent and more of the stresses obtained with the usual designing method as, for example, in the tests by Inge Lyse on slabs at Lehigh University[1] and those of the writer on rectangular beams[2]. The disagreement of the calculation with the actual data is only partly used by the method of Mr. Whitney[3] or that of Mr. Cox, as will be shown later.

*London, England.

(1) "A Study of Reinforcement in Concrete Slabs", Inge Lyse and George R. Wernisch, ACI JOURNAL, Sept.-Oct. 1936, *Proceedings*, V. 33, p. 1.

(2) "The Resistance of Reinforced Concrete Structures to Shock", by P. Abeles, *Concrete and Constructional Engineering*, Jan. 1942, p. 16.

(3) Discussion by Paul W. Abeles of "Plastic Theory of Reinforced Concrete Design", by Charles S. Whitney, *Proceedings* Society of Civil Engineers, June, 1941, p. 1131.

An excess of 50 per cent of the carrying capacity in slabs and rectangular beams with slight reinforcement is reason enough to study the matter more closely in order to make use of it in design. The concrete area in the tensile zone co-operates to some extent, which causes the tensile stress in the reinforcement between the cracks (especially in cases of small percentages) to be much lower than in the cracks. As the stresses at cracks increase those between the cracks increase gradually on both sides of the cracks, owing to the increased bond. Such an irregular straining pointed at intervals seems to be unlikely to agree with the reality. It may be expected that a redistribution of stresses takes place and the greatest steel stresses are smaller than those computed for the crack points. Besides, as long as the cracks are not too wide, it can be assumed that the actual steel stresses are materially greater than the yield point stress or its equivalent. Even if elongations of 5 per cent and more are taken into account (the equivalent for yield point straining is generally only 0.2 per cent and 0.4 per cent, respectively, for total and permanent elongation), no great harm can be caused if the elongation extends only to a small length. The latter depends on concrete tensile and compressive strength, on the distribution of the reinforcement, its diameter and behavior (strain-stress diagram), etc., to avoid wide cracks and to ensure a good bonding between the cracks. At failure, even stresses as high as the full strength may act in the steel at the cracks, as long as the latter have not surpassed a limit width. From all these considerations it will be realized that further investigation would be of the greatest importance in finding the basis necessary for design. These particulars must be dependent upon test results. Dr. K. Hajnal-Konyi[4] has examined such a proposition which agreed very well with the tests dealt with in Table D; but in this method only the compressive strength of the concrete is taken into account, and the results can therefore only correspond with the reality as long as a definite ratio exists between compressive and tensile strength, the bonding strengths also being of importance. That method, for example, disagrees with the tests shown in Table E.

Mr. Cox's formula is very simple. It seems, therefore, worth while extending this formula by considering an average concrete tensile stress f_{ct}, equally distributed over the tensile zone, according to Fig. C. The following two conditional equations (1) and (2) can be put up, $k = \frac{f_s}{f_c}$ and $kl = \frac{f_{ct}}{f'_c}$ denoting the ratios of the yield point stress and of the stress f_{ct} respectively and the concrete cylinder strength f'_c:

[4] Discussion by K. Hajnal-Konyi of "A Study of Reinforcement in Concrete Slabs", by Inge Lyse and George R. Wernisch, ACI JOURNAL, Jan-Feb. 1937; *Proceedings* V. 33, p. 16-1.

TABLE C

Tests of K. C. Cox on rect. beams 5 in. by 6 in. Comparison of some test results with theoretical values.

No.	100_p	f_s	f'_c	k	$\frac{M}{b.d^2}$ (psi)				$\frac{M_{test}}{M_{calc.}}$			k_1	Notes
		psi			tests	(a)	(b)	(c)	(a)	(b)	(c)		
211	0.76	56,000	3,300	17.04	434	373	400	431	1.11	6.08	1.01		
231	0.76	48,800		14.83	394	324	350	385	122	1.13	1.02	1	
321	0.88	53,200	4,800	11.09	516	406	445	493	1.27	1.16	1.03	60	
341	1.20	48,800		10.16	613	499	550	596	1.23	1.11	1.03		

TABLE D

Tests of Prof. Inge Lyse on Slabs 2 ft. 10 in. by 4 in. Comparison of test results with theoretical values.

No.	100_p	f_s	f'_c	k	$\frac{M}{b.d^2}$ (psi)				$\frac{M_{test}}{M_{calc.}}$			k_1	Notes
		psi			tests	(a)	(b)	(c)	(a)	(b)	(c)		
9	0.35	46,000	2,890	15.92	222	146	157	222	1.52	1.41	1.00		
10	0.303	55,300	2,795	19.66	226	153	163	226	1.48	1.39	1.00	1	
6	0.755	47,500	2,784	17.06	468	322	345	403	1.45	1.35	1.15	35	
7	0.754	50,000	2,640	18.95	469	330	349	401	1.42	1.34	1.16		
32	0.725	50,000	5,750	8.76	446	318	350	440	1.40	1.27	1.02	1/50	
22	1.32	48,500	3,140	15.45	632	543	576	630	1.16	1.10	1.00		
24	1.328	50,300	3,110	16.20	677	566	596	652	1.19	1.13	1.03	1/35	
12	0.343	95,500	3,390	28.20	392	298	312	384	1.31	1.26	1.02		
29	0.77	93,000	5,230	17.70	746	628	667	740	1.19	1.12	1.01	1/50	

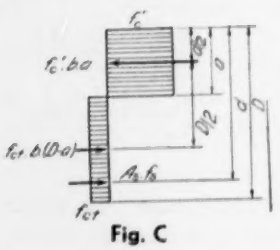

Fig. C

$$A_s f_s + f_{ct} b\,(D-a) = f'_c b a \quad \ldots\ldots\ldots\ldots (1)$$

$$M = A_s f_s\,(d - \frac{a}{2}) + f_{ct} b (D-a)\,\frac{D}{2} \quad \ldots (2)$$

$d_1 = \frac{D}{d}$ $a_1 = \frac{a}{d}$ and $p = \frac{A_s}{bd}$, thus the equations become:

$$pk + k_1\,(d_1 - a_1) = a_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1a)$$

$$\frac{M}{bd^2} = p\,(1 - \frac{a_1}{2})\,f_s + (d - a_1)\,\frac{d_1}{2}\,k_1 f'_c \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2a)$$

By putting the transformed equation (1b) : $a_1 = \frac{pk + d_1 k_1}{1 + k_1}$ into equation (2a) the following conditional equation can be derived:

TABLE E

Tests of P. W. Abeles on rect. beams 7.9 in by 9.1 in. Comparison of test results with theoretical values.

No.	100_p	f_s	f'_c	$k=\frac{4f_s}{3f^c_c}$ *	$\frac{M}{bd^2}$ (psi)				$\frac{M_{test}}{M_{calc.}}$			k_1	Notes
		psi			tests	(a)	(b)	(c)	(a)	(b)	(c)		
1	0.38	37,600	2,060	24.32	146	129	136	—	1.13	1.07	—	0	concrete 1, mild steel
2	0.85	37,100		24.02	300	273	283	—	1.10	1.06	—		
4	0.38	95,200	2,060	61.75	378	328	320	350	1.15	1.18	1.08	1/20	c. 1, high-grade steel
18	0.38	37,600		5.96	167	129	141	167	1.29	1.18	1.0		concrete 4
19	0.85	37,100	8,400	5.94	349	273	307	330	1.28	1.14	1.06	1/160	mild steel
20	1.50	37,100		5.94	581	468	534	555	1.24	1.09	1.05		
22	0.38	95,200		15.10	502	328	352	496	1.53	1.43	1.01		concrete 4,
23	0.85	87,000	8,400	13.80	911	642	696	826	1.42	1.31	1.10	1/25	high-
24	1.50	91,000		14.45	1375	1145	1217	1320	1.20	1.13	1.04		grade steel

*f_c is the cubic strength (cubes 7.9 in.).

TABLE F

Designing factors $\frac{M}{bd^2}$ for $f_s = 50{,}000$ psi. and $f'_c = 5{,}000$ psi.

100_p	(a) $n = 15$	(b) Cox	(c) formula (3)					
			$k_1 = 1/100$		$k_1 = 1/50$		$k_1 = 1/25$	
			1.1	1.3	1.1	1.3	1.1	1.3 d_1
0.2	93	99	128	140	156	179	211	241
0.4	181	196	224	235	251	274	304	349
0.6	266	291	317	329	344	366	408	439
0.8	349	376	410	421	435	457	484	527
1.0	430	475	500	511	524	545	571	614

$$\frac{M}{bd^2} = pf_s\left(1 + \frac{c_o}{pk} - cpk\left(1 + \frac{c_2}{pk}\right)^2\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

$$c_o = \frac{1}{2}d_1^2k_1\,,\ c_1 = \frac{1}{2(1+k_1)} \text{ and } c^2 = d_1k_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

In case $k_1 = 0$ $c_o = c_2 = 0$ and $c_1 = \frac{1}{2}$, thus the equation (3a) becoming identical with Mr. Cox's conditional equation 1.

The factor k_1 depends not only on the ratio of the concrete tensile and compressive strengths, but also on the property of the steel reinforcement. The concrete tensile strength varies between about 1/10 and 1/20 of the cubic strength (it is about 1/75 to 1/50 of the cylinder strength, assuming the latter to be ¾ of the cubic strength); the ratio 1/10 may be considered for concrete of comparatively small strength,

as is normally used, which ratio gradually decreases up to 1/20 for concrete of higher compressive strength. The behavior of the steel used also influences the factor k_1; for example, on the same concrete mild steel with a marked yielding causes a smaller excess than medium or high grade steel without direct yielding. The steeper the strain-stress diagram near the yielding, the greater the excess and the smaller the factor k_1. The latter varies therefore considerably, say from 1/20 to 1/200, the latter figure being assumed the limit for those cases in which the influence of the tensile zone is taken into account.

In Tables C and D some of the tests of Mr. Cox, Professor Lyse and the writer are investigated to have a comparison as to how far test results agree with the calculation, shown on $\frac{M}{bd^2}$ for case (a) calculation according to the usual designing method with $n = 15$ (these figures are only slightly altered if other values of n are considered), case (b) according to formula Cox and case (c) according to formula 3. The ratios $\frac{M_{test}}{M_{calc}}$ for these three cases show that according to formula Cox the excess is only partly reduced whereas the use of formula 3(a) gives a better agreement, provided a suitable factor k_1 is chosen.

Table C deals with four beams (211, 231, 321, 341) of Mr. Cox's tests, $d_1 = 4/3 = 1.33$ and k_1 assumed 1/50. Table D contains some of the test results on slabs of four inch depth of the tests of Professor Lyse, $d_1 = 6/5 = 1.2$, k_1 being assumed as 1/50 for specimens Nos. 29 and 32 with a cylinder strength over 5,000 psi and as 1/35 for all other specimens with f'_c smaller than 5,000 psi. Table E deals with some of the writer's tests on rectangular beams, concrete 1 being of very low strength and concrete 4 of high strength, each with two different grades of steel (mild steel and high strength steel), + of the cubic strength being assumed as equivalent to the cylinder strength, d_1 being 1.15. For concrete 1 and mild steel (specimens 1 and 2) the formula 3 has not been applied since the Cox formula agrees well with the test results on this poor concrete. For concrete 1 and high strength steel (specimen 4) $k_1 = 1/20$ has been considered, whereas for the high strength concrete 4 and mild steel a very small factor $k_1 = 1/160$ has to be assumed since the influence of the high strength concrete is relatively small owing to the yielding of the steel. For concrete 4 and high strength steel a factor $k_1 = 1/25$ agrees well with the test results.

Summing up it may be said that Cox's formula represents a simplification and improvement in the design of reinforced concrete construction. But with slabs and rectangular beams, slightly reinforced, the

concrete tensile zone apparently co-operates to some extent in spite of the discontinuity caused by cracks, thus favorably influencing the carrying capacity, especially when medium or high strength reinforcement material without marked yielding is used. Thus the Cox formula becomes less suitable for such cases. Since as shown an excess up to 50 per cent and even more is possible it seems advisable to consider this increased carrying capacity by extending the Cox formula to formula 3. For a certain quality of steel the factor k_1 for a concrete of usual average quality with a strength of f'_c (thus the factor k being known) could be found once and for all, on the basis of some tests on specimens of different percentages of reinforcement, whereas for each special concrete with extraordinary properties special preliminary tests, say on three different percentages have to be made in order to find that factor k_1 which gives the best agreement between calculation and reality for all those percentages for which the unextended Cox formula disagrees with the actual behavior.

Table F shows that if the ratios k and k_1 are known once and for all, the designing factors $\frac{M}{bd^2}$ for various percentages and different ratios d_1 can be computed, thus the design being made simple as with the usual method. As an example Table F contains the ratios $\frac{M}{bd^2}$ for some percentages up to 1 per cent of a steel of $f_s = 50{,}000$ psi and $f'_c = 5{,}000$ psi (thus $k = 10$). To show the influence of k_1 three different ratios 1/25, 1/50 and 1/100 have been considered and for any case two different ratios d_1 (1.1 and 1.3). For the purpose of comparison the respective factors according to the usual calculation method case (a) and according to Cox's formula case (b) have also been computed. From this table a considerable increase in the carrying capacity compared with the usual method of calculation, especially with small percentages and with a greater value k_1, can be seen. By making use of that increase a considerable saving of material would be possible. Further investigations in that respect are recommended.

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Job Problems and Practice

WHAT JPP MEANS

—an informal clearing house for mutual help on concrete problems; an opportunity for the ACI member who *knows* to tell the ACI member who *needs* the information. If the subject matter holds probabilities of general interest it will be briefed in these pages—with authorship credit unless the contributor prefers not.

If you know of an interesting problem whose solution has smoothed the way of someone in the field, record it briefly. If you have a problem, present it; a question, ask it. If you are moved to helpful comment or constructive criticism write the Secretary.

The "answers" are those of individuals to whom the questions are referred and not of the ACI.

Reinforcing for Concrete Masonry Walls (38-73)

Q—Some builders advocate the use of small deformed reinforcement bars in horizontal joints of walls of concrete masonry units. Where should this reinforcement be placed to be most effective in preventing the development of large cracks resulting from the shrinkage and temperature movements of the masonry?

BY W. G. KAISER*

A—Experience seems to indicate that the placing of reinforcing steel in horizontal mortar joints of concrete masonry walls gives them added tensile strength and distributes the stresses due to contraction and thereby tends to prevent the opening of cracks of a noticeable size. Usual practice is to use two ¼-in. pencil rods in the horizontal joints laying one rod on the outer and one in the inner face shell mortar bedding. The rods are run continuously around the wall with adjoining pieces lapping each other about 12 in. Such reinforcement is most effectively employed when placed in the horizontal joints immediately over lintels and under window sills. A special type of wall reinforcement has been developed using No. 9 gage inner and outer wires held the proper distance apart by diagonal cross wires welded to them. This type of reinforcement is easily placed in the proper position and is finding increasing use.

*Manager Cement Products Bureau, Portland Cement Association, Chicago.

Bars to Resist Bond Failures (38-74)

Q—What types of deformed bars are most effective in resisting bond failures?

BY HERBERT J. GILKEY*

A—Wernisch, in his American Concrete Institute paper (Mar.-Apr. 1938 JOURNAL; *Proceedings*, V. 34, p. 145) reporting tests conducted at Lehigh University, found that the diagonal types of corrugations appear to develop somewhat higher bond resistance than do the other commercial types. Both his tests and those of Abrams many years before, as well as tests reported by Posey, indicate that the fine textured, artificial and non-practical threaded bar develops a bond resistance much higher than can be developed by any of the commercial types of lugs. Moreover, the threaded bars do not have the same tendency to split the concrete as do the relatively large projections on regular deformed bars. The favorable behavior of the threaded type of bar seems to indicate that the development commercially of deformed bars with much finer and more closely spaced corrugations (a fine textured non-wedging type of roughness) would constitute a distinct advance over anything now commercially available. In further support of this idea it has been demonstrated that the presence of firm rust on reinforcement increases the bond.

Concrete Laundry Tray Manufacture (38-75)

Q—To the question: "What information is available on the manufacture of concrete laundry trays?" a competent ACI member gives the reply which follows.

A—This field of concrete products is essentially closed, with 20 to 25 manufacturers in the United States specializing in this type of product. The trays are shipped in carload lots and distributed through wholesale plumbing supply houses. The actual trade territory for an individual plant, due to carload shipments, is a 300- to 400-mile radius from the manufacturing plant.

The manufacturing methods incorporate a number of "trade secrets" which manufacturers guard jealously. Their attitude is that there are enough manufacturers of concrete laundry trays to take care of present requirements and a considerable increase in the market. Trays are now made and sold on a comparatively small margin based on volume sales. Two-unit trays 4 ft. long, 18 in. deep and 24 in. wide with two compartments of the simplest design are sold for from $7.00 to $9.00. Walls are usually built 1 in. to 1¼ in. thick and reinforced with No. 14 wire

*The question was answered by Professor Gilkey, who is Head Dept. Theoretical and Applied Mechanics, Iowa State College, Ames, at the Quiz Session of the 1941 Convention.

welded mesh reinforcement 2 in. on center both ways. In general, construction methods used are similar to that outlined in PCA data sheet: "Manufacture and Merchandising of Concrete Burial Vaults" insofar as concrete mixtures, methods of manufacture and curing are concerned. The market outlet varies almost directly with the number of housing units built in any particular trade area annually.

There are no specific specifications for this type of material and patented features are on extras rather than on the manufacture of the simple tray, such as metal inserts for the scrub boards and methods of attaching metal edging on the deluxe line of trays.

Simple Method of Estimating Percentage of Sand (38-76)

BY J. W. KELLY*

To supplement Stanton Walker's simple and useful method† of designing a concrete mix, with regard to his Step 5—determining the proportion of sand in the total aggregate—the following suggestions are offered. They are based on a variety of published and unpublished sources, including the writer's personnel obervations in the trial and calculation design of mixes with local materials in many cities.

The "optimum" percentage of sand for given materials, water-cement ratio, and consistency is that percentage for which the paste requirement (and hence the cement requirement) is a minimum. Since the paste requirement changes slowly at percentages of sand near the optimum, the value is not critical; an error of 2 in estimating the optimum percentage of sand increases the paste requirement only about 1 per cent, a negligible amount and within the tolerance of measurement.

In order to avoid possible harshness and honeycomb, it is well to use slightly more sand than the theoretical optimum, and in general to oversand rather than undersand.

Within the range of usual concrete mixes, the following relations appear to be fairly well established:

1. Gradation of coarse aggregate: For a fixed water-cement ratio and fixed consistency, increasing the maximum size of graded coarse aggregate from ¾ in. to 1½ in., or from 1½ in. to 3 in., permits the percentage of sand to be reduced by about 5.

2. Gradation of sand: Each increase of 0.1 in the fineness modulus of well-graded sand increases the required sand percentage by 1. Thus there is required about 3 per cent less of a fine sand, and 3 per cent more of a coarse sand, than of a medium sand.

*Assistant Professor of Civil Engineering, University of California, Berkeley.
†ACI JOURNAL, Jan. 1939, *Proceedings*, V. 35, p. 211.

3. Water-cement ratio: Each reduction of ½ gal. per sack permits the percentage of sand to be reduced by 1.

4. Consistency: A medium consistency requires 1 per cent less sand than either a stiff or a wet consistency; but this difference is so small that it need not be considered in design.

5. Crushed stone versus gravel: When the water-cement ratio is fixed, the optimum percentage of sand is about 3 higher for crushed stone than for gravel, and is about 3 higher for stone sand than for natural sand. When the cement content is fixed, the corresponding values are 5 and 5.

If the optimum percentage of sand has been established for one set of conditions, that for another set of conditions can be estimated with fair accuracy by applying corrections as indicated by the relations stated above. In the absence of data for the particular job, the following values will be useful as a starting point: For No. 4 to 1½ in. gravel, 0 to No. 4 natural sand, a medium consistency, and a water-cement ratio of 6½ gal. per sack, the percentage of sand, by weight, should be not far from 40.

A more specific simple method of estimation without the use of sieves or trial batches is based on the following relation: The optimum percentage of sand is practically always somewhat below the percentage for maximum density of mixed aggregate. The amount to be subtracted from the maximum-density percentage to obtain the practical optimum percentage is rarely as much as 8, and usually it will be of the order of 2 to 5; hence 3 can be taken for working purposes without error of consequence. None of the previous corrections, except perhaps that for water-cement ratio from a base of 6½ gal. per sack, need be made to the percentage of sand thus computed. The percentage of sand for maximum weight of mixed aggregate is easily obtained by determining (by A.S.T.M. C29 or similar method) the unit weight of dry aggregate mixtures containing 0, 30, 40, 50, 60, and 100 per cent of the sand, plotting a curve, and taking the value at the peak. For normal (not lightweight) aggregates, this value can be used regardless of the specific gravity of the aggregates; the *difference* between the peak value and the optimum value is affected but little as between maximum weight and true maximum density.

As in the case of any other proportioning method advocated to date, the estimated percentage of sand should be checked by trial either in advance of or on the job, by observing the mix and making such common-sense adjustments as may appear necessary.

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Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

The properties of the cements from the new Reich districts in the East

WILHELM EITEL, *Zement*, V. 30, Nos. 2, 3, Jan. 9, 16, 1941, p. 17-21, 29-32.
Reviewed by L. T. BROWNMILLER

Strengths, fineness data and chemical composition of the cements from the plants in Austria, Czecho-Slovakia and Poland are reported.

Studies in the system CaO-$5CaO.3Al_2O_3$-CaF_2

WILHELM EITEL, *Zement*, V. 30, Nos. 2, 3, Jan. 9, 16, 1941, p. 17-21, 29-32.
Reviewed by L. T. BROWNMILLER

Thermal studies in this system showed that the three compounds form a simple eutectic. No solid solution was found in the system. Tricalcium aluminate dissociates into CaO and $5CaO.3Al_2O_3$ in the presence of calcium fluoride.

How a cellular concrete bridge acts under traffic

F. W. PANHORST, *Engineering News Record*, V. 126, No. 25, (June 19, 1941) pp. 69-71
Reviewed by S. J. CHAMBERLIN

The cellular concrete bridge of 120-ft. span has counterweighted abutments to reduce moments to one-quarter of the normal amount. Of relatively low stiffness, the structure had observed deflections no greater than expected. Live load vibration is apparent but amplitudes are small. Measurements of deflection are to continue.

The development of bomb-proof structures

KURT KRAUSE, *Zement*, V. 30, No. 6, 7, Feb. 6, 13, 1941, p. 75-79, 88-92. Reviewed by L. T. BROWNMILLER

This article discusses principally the architectural features of bomb shelters. According to the author the present tendency is to build the structures above the ground in a style harmonious with the surroundings and in a form which may be utilized after the war. The shelters are built with a minimum of steel. The article is illustrated with a number of photographs but no construction plans are given.

Cantilever forms for Shasta Dam

Engineering News Record, V. 126, No. 23, (June 5, 1941), p. 81-83. Reviewed by S. J. CHAMBERLIN

With a 72-hour time limit for form removal and with 5-ft. concrete lifts on blocks, chiefly about 50-ft. square, cantilever forms with vertical studs carrying the concrete thrust without wales are proving advantageous. The panels are 50 ft. long and are designed for raising with three roller-chain jacks on A-frames. The space within the blocks is free from struts and tie-wires, and anchor bolts are only placed as each individual concrete pour nears completion.

Rapid method of determining alkalies in cement

FRANCIS W. GLAZE, *Rock Products*, V. 44, No, 7 July, 1941, p. 42. Reviewed by ROY N. YOUNG

A method for the determination of Na_2O and K_2O in cement is described. The time required is about one-third that for the modified J. Lawrence Smith procedure recommended tentatively by A.S.T.M. Committee C-1, without apparent sacrifice in accuracy.

The cement is decomposed by hydrofluoric acid in the presence of nitric and perchloric acids. The sodium is determined as uranyl zinc sodium acetate and the potassium from the platinum in the chloroplatinate precipitate.

A new approach to the problem of concrete wall reinforcing

F. N. ROPP, *Engineering News Record*, V. 126, No. 23, (June 5, 1941), p. 88-89
Reviewed by S. J. CHAMBERLIN

Observations of the structure and estimation of the shrinkage stresses in the second floor slab seemed to indicate that the cracks occurring at openings of the first floor concrete walls was due to the distortion of the building caused by slab shrinkage. The pull due to shrinkage of slabs would cause a variable unit soil resistance, which, together with uniform building load, would set up a variable shear and resulting moment in the walls. A system of diagonal wall reinforcing is suggested. Other suggestions are low-strength concrete and increased shrinkage reinforcement.

Steel floor slab forms produce "architectural" ceiling

Engineering News Record, V. 127, No. 1, (July 3, 1941), p. 60-62 Reviewed by S. J. CHAMBERLIN

A steel pan form for concrete floor slab construction is 12 in. wide, made of 14-gauge metal pressed into a U. Designed for an unsupported span of 10 ft., two-thirds of the conventional shoring is eliminated. The ends are supported by a transverse timber having a pressed steel spacer spiked to its top to keep the pans tight together. The under side of the concrete floor and beams are left exposed as the ceiling of the rooms below. The small radius along the longitudinal edges of the individual pan leaves a small fin which gives the appearance of a plank ceiling.

Placing and welding reinforcing steel in Pit River bridge piers

Engineering News Record, V. 126, No. 21, (May 22, 1941), p. 81-83 Reviewed by S. J. CHAMBERLAN

The long pieces of heavy reinforcing steel for high piers were held in place and supported by a central tower of scaffolding whose vertical members were second-hand 60-lb. T-rails. The scaffolding also supported the forms for interior cells and platforms, for workmen. The 2-in. steel bars, up to 60 ft. in length, were threaded down vertically into place upon a $\frac{3}{16}$-in. spacer plate resting on the bar below, the two adjacent ends fastened together by a clamp and the bars fastened at other points along their length by wire to 2 x 3-in. templates which were removed as the concrete level approached. The ends of the bars were cut at 45-degree angles and, in addition, the upper bar had the metal cut away to angle of 30 degrees. Prior to electric welding the adjacent bar ends were heated by gas burners to about 500°F.

Relationship of the garnet-hydrogarnet series to the sulfate resistance of portland cements

E. P. FLINT and LANSING S. WELLS, *Journal of Research of the National Bureau of Standards*, V. 27, No. 2, p. 171 (August 1941). Reviewed by P. H. BATES

Isometric $3CaO.Al_2O_3.6H_2O$, a constituent of set portland cement, is readily converted by sodium sulfate solutions to $3CaO.Al_2O_3.3CaSO_4.31.5H_2O$. Partial replacement of the Al_2O_3 in $3CaO.Al_2O_3.6H_2O$ by Fe_2O_3 or of the H_2O by SiO_2 gives "hydrogarnets" which are stable in the presence of sulfate solutions. Such substitutions may be brought about by inducing reactions between tricalcium aluminate hydrate and silica from various sources in the set cement or from added puzzolanic materials, and by increasing the content of glass and tetracalcium aluminoferrite in the cement.

Products having an X-ray structure nearly identical with that of grossularite garnet result when properly proportioned mixtures, made up of the puzzolanic material, dehydrated kaolin, with tricalcium silicate, or with lime and beta dicalcium silicate, are steamed at 500°C and 420 atmospheres.

The glass in portland cement

B. Tavasci, *Zement*, V. 30, Nos. 7, 8, Feb. 13, 20, 1941, p. 83-85, 100-103.
Reviewed by L. T. Brownmiller

A study was made of the changes in the celite (interstitial material) of Portland cement clinker, as the clinker was subjected to varying cooling rates from temperatures of 1500°C. These changes were followed microscopically by the reflected light method. H_2SO_4 was found to be the best etchant for this purpose.

By rapidly quenching very small clinkers in water, the celite was present as glass to a very limited depth around the outer edge of the clinker. In the inner portions of the clinker, the celite had a structure similar to that of the celite in air cooled clinkers. It was not present as a homogeneous phase, but as two distinct phases α- and β-celite. The α-phase corresponds to "dark interstitial matter"; the β-phase was not glass but probably a compound similar to tricalcium aluminate with alkalies in solid solution, possibly $8CaO.Na_2O.3Al_2O_3$. The relative amounts of these two phases depend on the rate of cooling. The conclusion is that true "glass" does not exist in commercial portland cement clinker.

Structure of hydrated portland cement

B. Tavasci, *Zement*, V. 30, Nos. 4, 5, Jan. 23, 30, 1941, p. 43-48, 55-56.
Reviewed by L. T. Brownmiller

A study was made of the micro-structure of neat portland cements using the method of reflected light. The specimens were impregnated with a tar-like substance and polished by the usual methods. Various etchants were used for distinguishing the phases present. By this method the author classified five different phases in the set cement. A number of photographs illustrate the results; these photographs are of the same high quality which have distinguished the author's former publications.

This paper represents one of the first publications on the structure of set cements by the reflected light method, which has proved so useful in the study of the structure of metals and cement clinker. Curiously, the method has been neglected in the study of our more important product—the set cement and concrete. Therefore this paper is recommended to our research institutions as another step for the solution of an extremely complicated subject. The method should be adopted in the study of concrete which is of even more importance than the structure of our intermediate product—the clinker.

More durable concrete with treated cement

M. A. Swayze, *Engineering News Record*, V. 126, No. 25, (June 19, 1941) p. 74-77.
Reviewed by S. J. Chamberlin

A report on the increase in resistance to frost action for concrete when (1) mineral oils, (2) animal or vegetable oils, and (3) natural resins are added to the cement in minute quantities. Mineral oil additions of 0.03 per cent had little effect on either workability or strength, but air voids were slightly increased and durability was increased by about 40 per cent (based on freezing cycles for 20 per cent loss). Beef tallow added in amounts of 0.03 per cent increased yield and caused a reduction in cement content. Lean mixes had little loss in strength. The air voids increased as the cement content was lowered, and increased with higher slumps. Shrinkage was increased, but the greatest effect was the increased durability, particularly for the lean mix range. Disintegration was uniformly distributed over all surfaces of the test specimens. Natural resins were added at the rate of from 0.01 to 0.05 per cent to constant proportions of

cement and aggregate with variable water content to produce the desired slump. Strength reductions due to increased air content ranged from 5 to 20 per cent. Resistance to freezing and thawing was markedly increased by the first 0.01 per cent increment of resin addition, with further increases with additional increments at a slower rate. Workability was increased, and observations indicated a decrease in permeability. It is thought that the increase in durability is due to the effect on the microstructure of the hardened paste, the capillaries being considerably smaller in size when the admixture is present, the concrete never becomes resaturated after drying out.

Investigation of special cements in the laboratory and in roads

Otto Graf and Kurt Walz, *Zement*, V. 30, No. 12, 13, 14, 15, 16, 17; Mar. 20, 27, Apr. 3, 10, 17, 24, 1941, pp. 153-158, 169-176, 181-186, 198-201, 205-210, 219-225. Reviewed by L. T. Brownmiller

Since 1936 the German specifications for cements for road construction require a relatively high transverse strength and low shrinkage. Generally cements for this purpose have a high iron oxide content. In 1938 a series of these cements were used in carefully controlled sections of the German auto highway between Berlin and Hanover.

The laboratory tests on the cements and the concrete specimens have been completed and are reported in great detail. The experimental road sections of the corresponding cements have been expowed approximately two years. There is some cracking evident in the road sections but no other signs of deterioration. Some of the cements show more cracking than others, but there is apparently no relation between the amount of cracking and the chemical composition. The high iron oxide cements do not show less cracking than the low iron oxide cements. Furthermore, there does not seem to be any relation between the extent of cracking and the transverse strength or shrinkage of the laboratory specimens.

The general conclusion at this date is that the high iron oxide cements show no advantage over the low iron cements. But the answer to the question will only be possible in several more years, since at least four years must expire before the behavior of the cements can be evaluated.

The laboratory tests reported here are pretty complete particularly with regard to the shrinkage and strength data. The concrete apparently was placed under carefully supervised conditions. No details of the method of manufacture of the cements are given.

Repeated stresses on reinforced concrete

F. C. Lea, *Struct. Engr.*, 1940, 18 (2), 511-20. Building Science Abstracts, V. XIII, No. 11, Nov. 1940, p. 169, London. Highway Research Abstracts

The investigation included measurements of the strain of three test beams before and after repeated loadings under maximum loads of 2.3, 4.5 and 5 tons. The strain measurements showed that the apparent modulus ratio of concrete beams depends upon the number of repetitions of the load and on movements such as creep and that it varies in beams as nearly alike as possible. From the results of the fatigue tests on seven reinforced concrete beams, which were subjected to loads up to 5 tons and up to 11 million repetitions it was found that the lowest load at which the beam will ultimately fail under repeated stresses is about 3½ tons, i.e., about 70 per cent of the static breaking load of the beam. (Editor: The "fatigue" limit of plain concrete is about 55 per cent of the static breaking load. Reference: H. F. Clemmer, "The Fatigue of Concrete", Proc., A.S.T.M. 1922, p. 408, and W. K. Hatt, "Fatigue of Concrete", Proc., Highway Research Board, V. 4, p. 47). An account is given also of tests of the effect of repeated and alternating stresses on the bond between concrete and steel, 2000 cycles per minute being used. The specimens consisted of ½-in. rods embedded to a depth of 3 in. in a

cylinder 3 in. in diameter made of a 1:3 cement-sand mortar. They were tested at 14 days in a Haigh fatigue testing machine; those of one series being subjected to repeated stresses from zero to a maximum, those of a second series to alternating stresses from equal tension to equal compression. In the first series of tests the lowest load at which the bond stress failed in fatigue was found to be about 300 psi. or about 50 per cent of the static bond stress. When the bond failed the rod drew out of the mortar. The lowest range causing failure was the same for the second series of tests and the rod again drew out on failure.

Swedish substitute cement to conserve coal

MOJE BERGSTROM, *Teknisk Tidskrift*, May 17, 1941. Reviewed by BENGT FRIBERG

For the yearly manufacture of about 570,000 tons of Swedish cement approximately 160,000 tons of hard coal have been required. With practically no coal mines in Sweden, extreme scarcity of fuel exists and it is clear also that the coal expected from Germany will not be obtained. The authorities have stopped the hard coal allotments to the cement industry and have asked that substitute cements be developed. Early in April all present stocks of standard portland cement were appropriated for government use and a week later a substitute cement, "E-cement" appeared on the market. The six manufacturers of E-cement guarantee the substitute cement as equal in weathering resistance to standard portland cement, class "A" cement. Compared to 28-day physical properties of class A-cement, compression 6000 psi. (cube test), and tension 500 psi., E-cement shows 3500 psi. and 350 psi., respectively. Based on accumulating experience, the rule has been established for smaller projects where pilot tests cannot conveniently be made, to use 35 per cent higher E-cement quantity than that specified for A-cement. Vibration is recommended wherever possible under qualified supervision.

All of the E-cements consists of a ground mixture of portland cement blast furnace slag and stone material. The second grinding of the portland cement in the mix results in a very fine grain size with increased effectivity of the cement portion.

There is no reason to assume that the water and weathering resistance of concrete will be decreased by the use of E-cement. Slag cement has been used extensively in other countries. In large Dam constructions in America, as well as in Norway, good results have been obtained with portland cements ground together with inert stone materials obtained near the place of construction. The sale of E-cement is not as yet restricted.

Reconstruction after the war

Editorial, *Concrete Building & Concrete Products*, (London), V. 16, No. 5, May, 1941, p. 73.
Reviewed by J. C. PEARSON

This editorial recalls the confusion in rebuilding after the World War, the scramble for materials and labor, and the soaring prices. It is still too early to develop a definite scheme for rebuilding after the present war, but it is none too soon to survey the resources of the country and to make plans of a general nature. It calls attention to the accumulation of exerpiences in selecting standardized designs for unit and pre-cast systems, and says that this will go far to offset the shortage of lumber, which will have to be imported. It recalls the struggle of the pre-cast industry 25 years ago for recognition in overcoming prejudices, and in the fallacy of emphasizing the adequacy of unskilled labor, which brought the material into disrepute. These teething troubles are now largely eliminated, quality is assured by reputable manufacture according to specifications, and the trade associations cooperate with the Government Departments in furnishing standard designs for specific purposes. Now there is a well-organized and well-equipped industry to aid in reconstruction.

At a recent meeting of the Institution of Structural Engineers, a well-known engineer had outlined the rebuilding problems. Assuming that there would be a shortage of bricks and seasoned lumber, but a ready supply of cement and steel, a unique opportunity would present itself for constructing more efficient dwellings. He went so far as to suggest that all post-war houses might well be of concrete and he described the essential details of such construction, embodying flat roofs, thin walls with adequate insulation, an air raid shelter nominally serving the purpose of a cellar, reinforced floors, communal heating, etc. He said that the concrete pre-cast units must be standardized as far as is reasonably practical; the architectural treatment must be suited to the construction and not vice versa.

Corrosive effect of inorganic fertilizers on concrete

ARTHUR F. PILLSBURY, *Civil Engineering*, V. 11, No. 6, June 1941, p. 348-349. Reviewed by J. R. SHANK

The effects of nitrogenous commercial fertilizers in irrigation water on concrete pipe were tested at the Citrus Experiment Station of the University of California at Riverside. Concrete pipe containers for several commercial fertilizers in three different concentrations and 2 x 4-in. concrete cylinders immersed in the solutions were the specimens used. Tests were also made using canal water, distilled water and solutions of sodium sulfate. The exposures were continued for one year. Solutions were renewed weekly for four weeks and monthly thereafter. Periodically the pipes and cylinders were inspected for visible corrosion. The cylinders were finally tested in compression. Differences in absorption characteristics of fragments of pipe immersed in the various solutions were also investigated. Studies were included as to the effects of the concretes on the *pH* of the solutions and vice versa.

Nine widely different cements of known compositions and physical properties were used. Commercial fertilizers used were ammonia (NH_3) (actually 28 per cent ammonium hydroxide); ammonium sulfate (20.5 per cent *N*); calcium nitrate (15.5 per cent *N*); "Calnitro" (20.5 per cent *N*) (ammonium nitrate with admixture of 35 per cent calcium carbonate); sodium nitrate (16.0 per cent *N*); and "Ammo-Phos" (16 per cent *N*, 20 per cent P_2O_5). Three concentrations were used, viz; 1 normal, (440 ppm nitrogen concentration), 4½ times normal, and 22½ times normal. One and 5 per cent solutions of sodium sulfate were used. Two acid solutions, 0.2 Grams H^+ and 5 Grams H^+ of sulfuric and nitric acids were also used.

The strengths of the cylinders from the different cements under the different treatments were not various excepting for the sulfate treatments and for one cement having high C_3A content and low specific surface where the strengths were down, and one with low C_3A and C_4AF contents where the strengths were high.

The strengths for all of the sulfate and "Calnitro" treated cylinders were well below the others and those treated with ammonium sulfate were particularly low. The "Calnitro" cylinders had strengths about two-thirds of those of the others. The sodium sulfate cylinders were much varied from no strength for the high C_3A, low specific surface cement, to 90 per cent of the others for the low C_3A, low C_4AF cement.

The different concentrations markedly affected only the "Calnitro" and sulfate treated cylinders.

Ammonium sulfate and sodium sulfate alone expanded the concrete.

Nitric and sulfuric acid did not affect the strengths.

None of the fertilizers excepting ammonium sulfate showed appreciable harm to concrete pipe lines when used in the customary manner.

In general, the *pH* of the solutions increased when they stood in contact with the concrete for a period of several days to a week.

Gasoline storage tanks of concrete

Teknisk Tidskrift, (Three Articles) Apr. 26, 1941. Reviewed by BENGT FRIBERG

"Tests and Practical Results for Under Water Storage of Gasoline in Concrete Tanks," by TOR. H. HAGERMAN, Ph. D.—Tests which have been under way in Sweden since 1938 for storage of oils and gasoline in water saturated concrete have given promising results. Further tests have been directed toward obtaining tanks of simple type and to investigate the effects of high pressures against the tank walls upon the stored fuel. The results have confirmed that little or no loss of gasoline occurs in storage. Laboratory tests with concrete in air showed capillary action of water three times as high as that of gasoline.

A recent concrete tank for 25,000 gal. capacity was constructed as a closed tank completely below the ground water level and anchored against uplift. Whether filled or not the walls are under pressure from the outside. Preliminary tests indicated a possible rate of infiltration as low as 15 gal. of water per year total. After four months' full storage of gasoline, the amount of water below the gasoline was 40 gal., but analysis of this water indicated that some of it may have come with the gasoline from earlier storage. The apparent gasoline loss for four months was 230 gal. ,the actual loss considerably less as correction must be made for temperature change of gasoline and tank.

"Construction of Concrete Tank for Storage of Gasoline," by T. B. HOST—The tank is of square cross section, 20 x 20 ft., 8 ft. 0 in. + inside height, with center stiffening walls in both directions, and a 5 cu. ft. sump at the center connecting all four compartments. A center shaft over the roof provides convenient access. The inside top of the tank is approximately 2 ft. below ground water level. For larger tanks the cylindrical form would be preferable.

The tank was separated from the solid rock base to permit circulation of water all around the tank. Pre-assembled form elements were used for each quarter section. Form bolts and tie wires were not permitted to extend through the concrete. After completed construction the tank was filled with water for the duration of the curing period. The concrete had a water-cement ratio of .57 as placed, 590 lb. of cement per cu. yd. concrete, and four different aggregate sizes to obtain a watertight concrete without hairline cracks.

"Concrete Tanks for Storage of Gasoline," by H. GRANHOLM—The author proposes a type of tank for storage of gasoline under outside water pressure for use where it is not convenient to locate the tank under an outside water surface.

The proposed tank construction consists of an outer section, dimensioned and reinforced to withstand inside water pressure, eventually outside soil pressure, and an inside section constructed to withstand outside water pressure. Between the two is a thin layer of a porous plaster or saturable mortar. Little reinforcement is required in circular inside walls. It will be convenient to construct the outside sections for floor and walls before the inside, and the reversed order for the roof, to permit the most convenient operations. Water from a small reservoir over the roof penetrates and saturates the porous plaster between the outside and the inside concrete shell, providing pressure against the gasoline inside the tank. In a central inspection shaft tank valves can be placed and the water pressure effectively controlled.

The practice of prestressed concrete

DR. PUSTON, *Beton und Eisen*, V. 39, No. 11, p. 150 (June 5, 1940) Reviewed by A. U. TREUER

The author describes a recent trend in the adoption of an entirely different set of statical combinations of steel and concrete from those conventionally used in reinforced concrete construction over the past forty years. Although neither new nor even unfamiliar, the adoption of these principles result in a unique type of structure.

The causes leading up to this development are discussed at length. Briefly, the recent advances in the development and availability of high strength yield point steel, the acute shortage of steel (Germany) and the difficulty of utilizing the special properties of high strength steels in conventional reinforced concrete design are the factors held responsible.

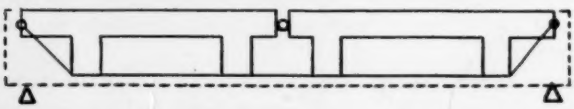
All of the structures described have this in common: they provide greater efficiency in the utilization of high strength steel in combination with concrete. The methods adopted for this purpose, however, naturally drop the structures into two distinct classes:

(1) Those where steel is introduced into a monolithic unit while held in a stressed condition. (This type of construction has been adequately dealt with in past literature under the general heading of prestressed concrete.)

(2) Those in which the steel is taken entirely out of the cross-section of the compressively stressed concrete so that it may be influenced in an independent fashion, and either due to the method adopted in construction or by artificially applied forces stressed in tension without a similar stress being introduced in the surrounding concrete.

In its simplest form the trussed beam will serve as a sample of the type. It is here perhaps that the article's principal interest lies. Data, as well as references to a number of structures built during the past six years, are given.

To this reviewer the most interesting structure described is the Wiedenbruck bridge shown by a schematic sketch below:



The introduction of a hinge near the center of the compression member or chord results in a statically determinate structure. (The pre-stress in the steel is introduced by way of controlled centering.) The quantity of steel required was but one-third that required for a comparable all-steel structure. According to the designer, freely supported beams of the type may be economically built for spans up to 230 ft. and a depth of 17.0 ft.

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Proposed Recommended Practice for Measuring, Mixing and Placing Concrete*

Reported by ACI Committee 614

LEWIS H. TUTHILL
Chairman

JOHN G. AHLERS
CHARLES F. BALL
E. W. BAUMAN
SAM COMESS
R. V. CREPPS
LION GARDINER
BYRON A. HILL
ARTHUR A. LEVISON
O. G. PATCH
DONALD D. REYNOLDS
R. W. SPENCER
HARRY F. THOMSON
IVAN L. TYLER
RODERICK B. YOUNG

I. INTRODUCTION

1. This report has been prepared in response to an expressed need for an outline of good practices for measuring and mixing the ingredients for concrete and for placing the finished product.

2. At the outset the Committee faced the question whether it should describe "common practices" or "best practices." Since this report is a recommendation, and not a specification, it is believed that the interests of progress in the development of concrete construction will be better served if the best methods known are outlined.

3. On small jobs, or in the more competitive uses of concrete such as building construction, practices somewhat more relaxed than those

*A report of Committee 614 was published in ACI JOURNAL, Feb. 1940, *Proceedings* V. 36, p. 329, released by the Standards Com. as information and for discussion (see Sept. 1940 Supplement, p. 352-1). In closing the preliminary discussion the Committee proposed revisions of the original report. Subsequent consideration in the committee developed other revisions. In the present revised form (received by the Institute Oct. 15, 1941) the report is released by the Standards Committee for Convention consideration on a proposed motion for adoption as an ACI standard. The report has been approved by 13 of the 15 members of the Committee, two members not voting.

For convenience in considering the present, revised draft of a former report note that the paragraphs of the present draft which are new or largely so or which contain revisions from the preliminary report are indicated in the following list of paragraph numbers by affixing "R" for revised or "N" for new.

I—Introduction: 4 and 5R.
II—Measurement: 2, 3 and 9R.
III—Mixing: 1, 2, 4, 7, 11, 13, 14 (including (b) and (c))R; 15, 16 and 17N; 18 and 20 R.
IV—Placing: 3, 7 and 8 R; 9N.
V—General Considerations: 1R; 4, 5, 6 and 7N; 8 and 9R.

here recommended may be economically justified. The profitable degree of such deviation must be determined by those in charge of each job, in view of the conditions which obtain. In general, the Committee has no evidence that good concrete operations cannot be performed as economically as poor ones or that the methods here recommended, where applicable, will result in higher ultimate costs than other methods. The refinements recommended are intended primarily to improve uniformity and to eliminate segregation in aggregates and concrete. Where properly executed, the methods described have resulted in smoother operation and any additional costs resulting from them have been offset by consequent higher production rates.

4. It has been assumed that whoever will give serious consideration to these recommendations will have a reasonable knowledge of the ordinary practices and preparations required in concrete work. For that reason, many routine instructions for measuring, mixing, and placing concrete are omitted. Since the specific objective of these recommendations is maximum uniformity, homogeneity and quality of concrete in place, special consideration is given practices designed to accomplish that end. In order to portray more clearly certain of the principles involved and their application to concreting operations, four figures are included as a part of these recommendations, illustrative of some examples of good and bad practice.

5. For further discussions of the standard aspects of measurement, mixing, and placing, the reader is referred to the 1940 Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete and to the particularly excellent and specific document of recommended practice adopted by the American Association of State Highway Officials relating to Measuring, Mixing, and Placing of Concrete for Use in Concrete Pavements. With the recommendations of those reports, ACI Committee 614 is, in general in accord, although a few exceptions in details or in emphasis may be noted.

II. MEASUREMENT

Essential requirement

1. Equipment should be used which is capable of performing accurate measurement. Materials should be handled, and measuring operations performed in such a manner that satisfactory reproducibility of the selected batch assembly and aggregate grading is obtained batch after batch. The objective is ultimate uniformity and homogeneity of the concrete.

Aggregates

2. The number of separate sizes of coarse aggregate which should be used, to reduce segregation in handling to a practical minimum and

to permit reasonably close control of the grading from batch to batch, is dependent upon the maximum size of the material. For particles larger than 1 inch, the ratio of the maximum to the minimum designated sizes should not exceed 2; for coarse aggregate particles finer than 1 inch, this ratio should not exceed 3. The permissible tolerance of undersize or oversize material in each designated size of aggregate should not exceed 10 per cent, and handling operations should be such that variation in these portions of such material is held to a practical minimum.

3. The maximum size of sand should not exceed that which will pass a screen having ⅛-in. openings if it is screened and handled dry. The standard No. 4 sieve (3/16-in. openings) represents a suitable maximum size for wet screened sand which is to remain damp throughout all handling and batching operations, except when the use of the smaller size is desirable to control the percentage of pea gravel (about No. 8 to ⅜-in.) in the mix. When two sizes of sand are used to obtain proper grading, no attempt should be made to blend them by placing alternate amounts in stock piles or in cars and trucks as loaded or by other equally crude procedures. With adequate equipment and ample supervision and inspection, good results are obtainable, when the fine aggregate is produced, by blending the different sizes of sand as they flow into a common stream from regulating gates or feeders. Such methods can, however, produce very irregular results. For the majority of work, separate batching of the fine and coarse sand is preferable since it will produce consistently uniform results under the wide range of common plant and job conditions.

4. The production or purchase of sand should be controlled insofar as possible to minimize variations in grading, particularly in the finer sizes. In plants where sand is washed, care should be exercised to avoid removal of fines essential to good grading and to plastic, workable concrete. For sand passing a ⅛-in. screen, 20 per cent finer than 50 mesh is considered close to an ideal percentage of fines for average concrete work. Lean mixes are benefited by ample percentages of such fines while rich mixes are improved with a smaller percentage of these sizes.

5. Varying amounts of water in aggregates commonly contribute to lack of uniformity. Effort should be made to maintain a uniform moisture content in the aggregate supplied for batching. For this reason aggregates from dry deposits, if not high in absorption, may preferably be prepared and handled dry when washing is not essential. Before placing the material in batcher bins, wet aggregates should be drained until, by actual test, there is little difference in the moisture content of the wettest and driest portions of the materials. Adequate drainage

will require at least 8 hours and additional time may be necessary, depending on the grading of the material and the efficiency of the drainage system provided. On large jobs when thorough drainage is difficult, an electric moisture indicator in the sand batcher and a consistency meter on the mixer will permit rather close control of consistency, even when there are wide fluctuations in aggregate moisture.*

6. Aggregates should be handled in stockpiles and bins in a manner producing minimum segregation of sizes. Stockpiles should be permitted only when built up in layers no thicker than those resulting from truck loads dumped on the same plane. When stockpiling sand and pea gravel, segregation may be further avoided if moisture contents of at least 5 and 2 per cent, respectively, are maintained. Coarse aggregate should be removed from stockpiles, by clamshell bucket or other means, in horizontal layers of such width that aggregate is not permitted to run down the slopes at the edge of the pile. Storage bins should have the smallest practicable equal horizontal dimensions. Round bins are preferable. Bins should have only one outlet located at the center of a bottom sloping to it from all directions at not less than 50 degrees from the horizontal. They should be charged by material falling vertically directly over the outlet. They should be kept as full as possible at all times to avoid breakage and grading changes as the bins are emptied.

7. Reference is made to the sketches, a part of these recommendations, for illustrations of preferred practice in the handling of aggregates. Unless precautions are exercised in the selection and handling of aggregates, refinements in measuring, mixing and placing are nullified.

Batching aggregates

8. *For jobs of from 2,000 to 5,000 cu. yd.*, the cost of hand batching soon justifies a simple weigh batcher installation, into whose bins aggregates may be delivered directly. As the size of the job increases, the saving in materials and the prevention of contamination by dirt or by other sizes adds considerably to the balance in favor of bins as opposed to dumping materials on the ground. As the size of the work increases still greater refinement should be required.

9. *From 2,500 to 10,000 cu. yd. jobs*, cumulative manual weigh batching and from *10,000 to 25,000 cu. yd.*, cumulative automatic weigh batching are recommended. For jobs of over *25,000 cu. yd.*, requirement of individual automatic weighing of the aggregate is justified by the improved perfection of uniformity of measurement. (See favored sketch.) If good inspection and dependable plant men are available,

*Engineering News-Record, Page 52, July 11, 1935.

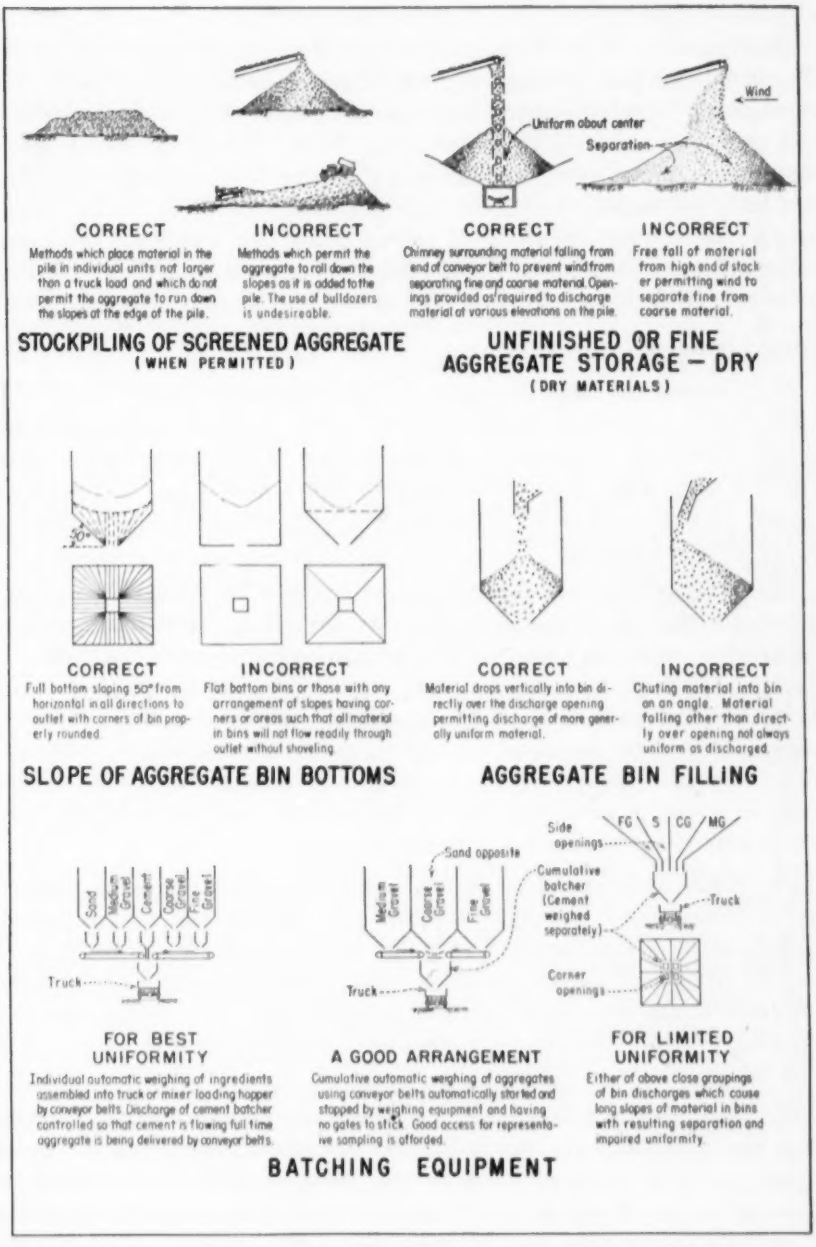
CORRECT
Methods which place material in the pile in individual units not larger than a truck load and which do not permit the aggregate to run down the slopes at the edge of the pile.
INCORRECT
Methods which permit the aggregate to roll down the slopes as it is added to the pile. The use of bulldozers is undesireable.
STOCKPILING OF SCREENED AGGREGATE
(WHEN PERMITTED)
Uniform about center
Separation
Wind
CORRECT
Chimney surrounding material falling from end of conveyor belt to prevent wind from separating fine and coarse material. Openings provided as required to discharge material at various elevations on the pile.
INCORRECT
Free fall of material from high end of stacker permitting wind to separate fine from coarse material.
UNFINISHED OR FINE AGGREGATE STORAGE – DRY
(DRY MATERIALS)
50°
CORRECT
Full bottom sloping 50° from horizontal in all directions to outlet with corners of bin properly rounded.
INCORRECT
Flat bottom bins or those with any arrangement of slopes having corners or areas such that all material in bins will not flow readily through outlet without shoveling.
SLOPE OF AGGREGATE BIN BOTTOMS
CORRECT
Material drops vertically into bin directly over the discharge opening permitting discharge of more generally uniform material.
INCORRECT
Chuting material into bin on an angle. Material falling other than directly over opening not always uniform as discharged.
AGGREGATE BIN FILLING
Sand
Medium Gravel
Cement
Coarse Gravel
Fine Gravel
Truck
FOR BEST UNIFORMITY
Individual automatic weighing of ingredients assembled into truck or mixer loading hopper by conveyor belts. Discharge of cement batcher controlled so that cement is flowing full time aggregate is being delivered by conveyor belts.
Sand opposite
Medium Gravel
Coarse Gravel
Fine Gravel
Cumulative batcher (Cement weighed separately)
Truck
A GOOD ARRANGEMENT
Cumulative automatic weighing of aggregates using conveyor belts automatically started and stopped by weighing equipment and having no gates to stick. Good access for representative sampling is afforded.
Side openings
FG
S
CG
MG
Truck
Corner openings
FOR LIMITED UNIFORMITY
Either of above close groupings of bin dischargers which cause long slopes of material in bins with resulting separation and impaired uniformity.
BATCHING EQUIPMENT

Fig. 1—Correct and incorrect handling of aggregates in storing and batching

it may not be economically justifiable to require new equipment, especially for the smaller jobs in the foregoing brackets.

10. Individual automatic and cumulative manual aggregate batchers should be charged through easy operating clamshell or undercut radial type gates. Power-operated gates are used with the automatic batchers and may be used with the manual batchers. Where necessary for the required degree of accuracy, the gate charging the automatic equipment should be arranged to operate with a suitable "dribble" after there is nearly the desired quantity of material in the batcher. Automatic cumulative batchers should preferably be supplied from short conveyor belts operating between the center of the bin bottom and the batcher.

Batching cement

11. Cement for large jobs should be handled in bulk and weighed for each batch, preferably automatically. New equipment for this purpose probably is not justified except on jobs of well over 10,000 cu. yd. Cement should never be weighed with the aggregate. Sacked cement should be measured in units of not less than one sack unless the fractional bags are weighed.

12. Bulk cement batchers should be charged by means of controlled screw conveyors or equally effective devices. Moisture should be thoroughly removed from the air used in jets to loosen the cement in storage. Cement hoppers and batchers should have smooth metal or metal covered sides with rounded corners. Silos, bins, and hoppers for cement should be governed by principles outlined above for aggregate bins. Control should be available on the batcher discharge gate so that, in event of over-run, the excess cement may be withheld unless it is removed before the batch is discharged.

13. Modern specifications issued by government and highway agencies contain excellently detailed specifications for manual and automatic weigh batching equipment for cement and aggregate. Ordinarily, the present equipment on the market, when in good mechanical condition, will have approved tolerances in weighing ability. Operation should be required within a degree of accuracy of 1 per cent for cement and 2 per cent for aggregate.

14. Bulk cement is often lost or scattered indescriminately in dropping from the batcher to trucks or batch cars below. Free fall of cement should never be permitted. A typical good method of preventing this is to enclose the discharging cement in a narrow canvas boot long enough so that its outlet may be buried in the cement when loading portable batch compartments. Such compartments should contain a separate section for the cement, attached to and operating with the individual

batch release gate. The boot may be used as a tremie or the cement may be eased into the compartment by controlling the amount of kinking or doubling back of the canvas at the lower end of the boot. The canvas may then be lifted, shaken, and dropped into the next compartment. A telescopic rubber hose drop-chute from the cement batcher is also available for this purpose. If a separate section is not provided for the cement in each batch compartment, or the cement is not enclosed in the aggregate, tarpaulin covers should be required during transportation. For placing bulk cement into a batch hopper for a stationary mixer, a pipe of suitable size to hold the cement should extend from the batcher discharge to a level near the bottom of the hopper. After covering the end of this with the aggregate, the cement may be discharged. It will then enter the mixer without loss of dust, and it will be well distributed through the entering aggregate.

Water measurement

15. Water measurement on the larger jobs justifies the recommendation that for pavers and other portable mixers, there be used only automatic meters of proved mechanical dependability, or automatic measuring tanks of the vertical cylinder center-siphon discharge type, capable of routine measurement within an accuracy of 1 per cent under all operating conditions. For central mixing or batching plants, either of these devices or automatic weigh batchers of proved dependability and accuracy within 1 per cent should be required. Tanks should be permitted only as an auxiliary part of the automatic meter or weigh batcher, but not as a means of measurement, except in the form of a vertical cylinder with center-siphon discharge.

The small jobs

16. For the small job, usually less than two to five thousand cubic yards, concreting operations are likely to be so irregular that many of the foregoing preludes to uniformity in the concrete, as far as aggregate preparation and handling are concerned, are often next to impossible of attainment. It is well to keep them in mind, however, as standards to be used wherever possible. Cement should be handled and measured in sacks. Mixers should be of sufficient capacity and batches of such a size that no fractional bags of cement are required unless they are weighed for each batch. Water measuring equipment of expensive refinement may not be economical for the small job, but the use of simple equipment that is accurate, positive and dependable is justified and should be required. A manually operated standard disc water meter, of capacity according to the size of the mixer, equipped with a vertical-face, set-back register is not expensive and is recommended where water temperatures do not exceed 100°F. A vertical cylinder tank with center-

siphon discharge is equally satisfactory, and should be used if water temperatures are greater than 100°F. Aggregates should be weighed. In a metropolitan area where weigh batched aggregates or plant controlled ready mixed concrete is available, one of these should be used, even at a reasonable premium, in preference to handling and measuring the materials on the job. Where such services are not available, the quantities for each batch should be weighed in wheelbarrows or buggies on platform scales now generally available for this purpose on small jobs.

General

17. If standards of uniformity, such as are readily obtainable with modern batching equipment, are to be maintained uniformly throughout an operation, there is greater need for emphasis of correct procedure in incidental operations than on the actual weighing of ingredients. Care should be exercised that batches accurately assembled arrive as uniform batches in the mixer. These sources of error should be avoided: (1) Over-lap of batches in loading and discharging multiple batch trucks and cars. (2) Loss of material in transferring batches to the skips of portable mixers. (3) Loss or "hang up" of a portion of one batch, or its inclusion with another, when dry batches are transferred by means of belts and hoppers. All such damage to the correctness of the quantities in each assembled batch should be strictly avoided and methods should be approved or discarded accordingly.

III. MIXING

Essential requirements

1. Thorough mixing is, of course, a first essential. Equipment and methods used shall be such as to insure uniformity of strength, and uniformity of consistency, cement and water content, and aggregate grading from beginning to end of each batch *as discharged*. For concrete work of the highest general quality, mixing equipment should be used which is capable of handling concrete suited to the particular work, from the standpoint of its placeability and workability *in the forms* (preferably by means of vibration where this is applicable). Mixing equipment requiring, for efficient operation, mixes containing larger proportions of sand, cement, or water than indicated for best results on the basis mentioned is not recommended.

The mixer

2. The mixer should preferably have a combination of blade arrangement and drum shape such as to insure an end to end exchange of the materials parallel to the axis of rotation as well as a rolling, folding, or spreading movement of the mix over on itself as the batch is mixed. Otherwise the action should be such that comparable mixing efficiency

and uniformity of the batch is obtained within a reasonable mixing period. The mixer should be operated at the speed designated by the manufacturer, unless a reasonable change of speed definitely shows better results.

3. Any mixer leaking mortar or causing waste of materials due to faulty size, shape, or operation of charging equipment should be taken out of service immediately until satisfactory repairs and improvements have been made to eliminate such waste. Hardened concrete or mortar should not be permitted to accumulate on the inner surfaces of mixing equipment. Worn blades should not be continued in service when they materially decrease mixing efficiency.

Size of batch

4. The size of the batch in job mixers should not exceed the manufacturer's guaranteed capacity of 10 per cent more than the rated capacity of the mixer. The rated capacity should be exceeded only when: (1) it results in no loss of mortar during mixing, and (2) tests show the larger batch to be properly mixed. When, to shorten time of discharge or to reduce segregation due to retention of coarse aggregate, it is desired to retain a portion of the batch in excess of the normal "hold back," the amount of material held back should be taken into account in fixing the size of the batch in relation to the capacity of the mixer.

The charging operation

5. All ingredients of the mix should enter the mixer promptly and as uniformly as possible at the same time. It is particularly important in charging stationary mixers that the solid materials be arranged in the charging hopper in such a manner that no one of them enters separately but that proportional amounts of each (particularly of cement and sand) will be in all parts of the stream of material as it flows into the mixer. To avoid "gumming" and for greater uniformity, cement should never be charged separately, especially ahead of the aggregates but should be enclosed within the other materials. Water for the batch should be released first and continue to flow while the solids are entering the mixer, and should have completed flowing shortly after the last of the batch has entered the drum. This flow should not continue for more than the first 25 per cent of the mixing time, and should be stopped as much short of this as the apparent requirements of good operation of the particular mixer will permit. Where meters are used, and particularly if water pressure is low, auxiliary tanks, into which the quantity of water for the batch may be measured in advance, equipped with a discharge pipe to the mixer of such a size as to insure the foregoing rate of delivery, may be necessary.

Mixing time

6. The net mixing time for a one-yard or smaller mixer when mixing gravel concrete of medium consistency containing 1.5 bbl. or more of cement per cu. yd. should not be less than 1 minute under the best conditions. For concretes containing less cement, mixed to drier consistencies, or containing an unusually harsh mixing aggregate, time should be increased 25 per cent to 50 per cent according to the degree of departure from concretes of the most readily mixed type.

7. For job mixers larger than one cu. yd. capacity, particularly when mixing mass concrete, a nominal minimum net mixing time of 1 minute plus ¼ minute for each additional cubic yard of capacity should be required. The use of mixers unable to pass subsequently prescribed tests for mixing efficiency after twice this period of mixing and when charged indescriminately, should be prohibited. The total net mixing time for mixers meeting this test should be the foregoing minimum (determined by the size of the mixer) plus such additional time as necessary to bring the batch within the requirements of the mixer efficiency test when the mixer is charged and operated in accordance with practice which may be established as routine for the job.

8. The mixing period should be measured from the time when all the solid materials are in the mixer drum provided that all the mixing water shall be introduced before 25 per cent of the mixing time has elapsed. Over-mixing of more than three times the above required periods should not be permitted and for that reason all mixing equipment should be so arranged that it can be stopped and started under full load.

9. A batch timer and counter, including an automatic lock which will release the discharge lever only at the end of the proper mixing period, and an audible indicator, should be installed and maintained in operation on each concrete mixer. On truck mixers the audible indicator is unnecessary. For jobs requiring 2 cu. yd. or larger stationary mixers a recording watt meter or consistency meter has definite advantages and should be required on each mixer because it is more reliable as a batch counter, and because it provides a permanent record of number of batches, type of batch, mixing time, retempering, and the uniformity of consistency. Failure of any of this equipment should be sufficient cause for discontinuance of the use of the mixer until the device is repaired or a new one is installed.

Retempering

10. Retempering or indiscriminate addition of water to delayed batches which have become stiffer than the usual consistency should be

prohibited. Under careful supervision an increment of water may sometimes be added with results beneficial to essential workability, provided the maximum allowable water-cement ratio is not exceeded. Concrete delayed in placing should not be wasted unless the amount of additional water, necessary to restore sufficient workability to permit it to be placed, would exceed the allowable water-cement ratio. Batches considerably wetter than the specified consistency, or batches in which there has been some obvious error in the measurement of solid ingredients, particularly a shortage of cement, should be wasted unless proper corrections can be made at once and the corrected batch thoroughly mixed.

Ready mixed concrete

11. The use of ready mixed concrete is an important development in connection with general concrete construction—particularly in metropolitan areas. A ready mixed concrete operation, adequately equipped and supervised, affords excellent opportunities for the control of concrete quality. Thus, ready mixed concrete may be made to offer the small job the control facilities ordinarily available only to large jobs.

12. Ready mixed concrete may be centrally mixed, truck mixed (mixed in transit), or partially centrally mixed and partially truck mixed (often referred to as "shrink-mixing"). The use of ready mixed concrete introduces certain new problems of control to which, while not fundamentally different from those encountered in other forms of concreting operations, attention should be directed.

13. Centrally mixed concrete presents about the same problems as job-mixed concrete which must be transported for considerable distances from the plant to the place of deposit. The possibilities of loss in consistency, resulting from lapse of time between mixing and placing, should be given consideration. For normal periods and temperature, and when agitator trucks are used for transportation, this factor is generally negligible. Nevertheless, precautions should be taken to minimize loss of slump by expediting delivery, eliminating delays and, in warm weather, keeping the concrete as cool as practicable by using cold mixing water, avoiding hot cement, and by shading and sprinkling aggregate. Under conditions where loss of workability cannot be otherwise controlled and is likely to be a serious matter, mixing at the forms or adequately controlled truck mixing as hereinafter outlined, will eliminate the difficulty.

14. In the case of truck mixing operations it is necessary to exercise more than usual control of all factors governing consistency because of the difficulty of judging consistency until the concrete is delivered and discharge is started. Variations in consistency may be minimized by:

(a) Handling aggregates in such a manner as to reduce variations in grading and moisture content to the greatest practical extent.

(b) Exercising accurate control over the quantity of mixing water which may be admitted to the truck mixers.

(c) Telephone communication between the point of placing and the batching plant to keep minimum the quantity of concrete arriving before a change order goes into effect.

(d) Responsible technical supervision of the entire operation.

15. In some cases satisfactory control may be readily obtained only by withholding the final increment of the mixing water until the mixer arrives at the form and then doing the additional mixing as necessary to incorporate thoroughly the added water. As pointed out earlier, complete truck mixing en route will produce consistencies of satisfactory uniformity, with minimum water content, only when special care is exercised in the control of grading and moisture content; in the absence of such control resort should be made to mixing at the forms or to completing the mixing at the forms. While every effort should be made to have concrete batches arrive at the job at a suitable consistency, it is possible to make adjustments after arrival if proper provisions are made in specifications and on the job. If the batch arrives too dry, but contains less water than the maximum allowed, additional mixing water, within specification limits, may be added and additional mixing done at the job site. In general, batches which arrive too dry, and already contain the maximum allowable mixing water, or batches which arrive too wet, should be rejected. However, adjustments may be made where conditions permit, although they are to be avoided since they interfere with orderly routine. Every care should be exercised to see that concrete arrives at the job with the desired consistency.

16. Truck mixers should be operated at a mixing speed in accordance with the recommendations of their manufacturer and should also be capable of operating at a suitable lower agitating speed of not less than 2 nor more than 6 r.p.m. of the drum. The time of mixing in truck mixers should be not less than 50 revolutions of the drum or blades at the rate of rotation specified by the manufacturer as mixing speed. The volume of the batch should not exceed the manufacturer's rating of the truck mixer's capacity and, in any event, should not be greater than fifty-seven and one-half (57.5) per cent of the gross volume of the drum or container. If, from any cause, this amount of mixing is insufficient, as determined by the subsequent tests for mixer efficiency or by other means, additional mixing should be required until satisfactory results are obtained. In truck mixing and shrink mixing, not more than 150

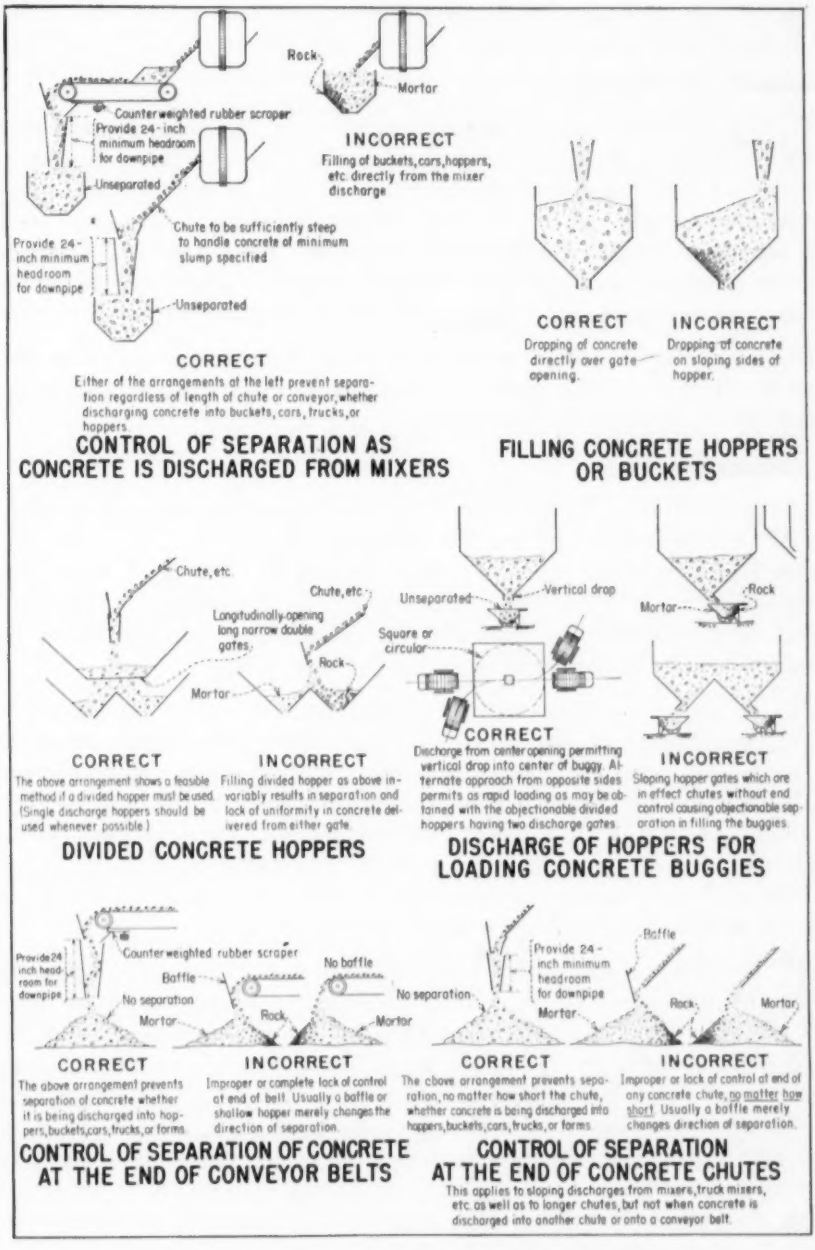
Rock
Mortar
Counter weighted rubber scraper
Provide 24-inch minimum headroom for downpipe
Unseparated
INCORRECT
Filling of buckets, cars, hoppers, etc. directly from the mixer discharge
Chute to be sufficiently steep to handle concrete of minimum slump specified
Provide 24-inch minimum headroom for downpipe
Unseparated
CORRECT
Either of the arrangements at the left prevent separation regardless of length of chute or conveyor, whether discharging concrete into buckets, cars, trucks, or hoppers.
CONTROL OF SEPARATION AS CONCRETE IS DISCHARGED FROM MIXERS
CORRECT
Dropping of concrete directly over gate opening.
INCORRECT
Dropping of concrete on sloping sides of hopper.
FILLING CONCRETE HOPPERS OR BUCKETS
Chute, etc.
Chute, etc.
Longitudinally-opening long narrow double gates.
Rock
Mortar
CORRECT
The above arrangement shows a feasible method if a divided hopper must be used. (Single discharge hoppers should be used whenever possible.)
INCORRECT
Filling divided hopper as above invariably results in separation and lack of uniformity in concrete delivered from either gate.
DIVIDED CONCRETE HOPPERS
Unseparated
Vertical drop
Square or circular
CORRECT
Discharge from center opening permitting vertical drop into center of buggy. Alternate approach from opposite sides permits as rapid loading as may be obtained with the objectionable divided hoppers having two discharge gates.
Rock
Mortar
INCORRECT
Sloping hopper gates which are in effect chutes without end control causing objectionable separation in filling the buggies.
DISCHARGE OF HOPPERS FOR LOADING CONCRETE BUGGIES
Provide 24 inch head-room for downpipe
Counter weighted rubber scraper
Baffle
No separation
Mortar
Rock
No baffle
Mortar
CORRECT
The above arrangement prevents separation of concrete whether it is being discharged into hoppers, buckets, cars, trucks, or forms.
INCORRECT
Improper or complete lack of control at end of belt. Usually a baffle or shallow hopper merely changes the direction of separation.
CONTROL OF SEPARATION OF CONCRETE AT THE END OF CONVEYOR BELTS
No separation
Provide 24-inch minimum headroom for downpipe
Mortar
Baffle
Rock
Mortar
CORRECT
The above arrangement prevents separation, no matter how short the chute, whether concrete is being discharged into hoppers, buckets, cars, trucks, or forms.
INCORRECT
Improper or lack of control at end of any concrete chute, no matter how short. Usually a baffle merely changes direction of separation.
CONTROL OF SEPARATION AT THE END OF CONCRETE CHUTES
This applies to sloping discharges from mixers, truck mixers, etc. as well as to longer chutes, but not when concrete is discharged into another chute or onto a conveyor belt.

Fig. 2—Correct and incorrect methods of handling mixed concrete

revolutions of the drum should be at a peripheral speed in excess of that designated as agitator speed. Additional mixing, if necessary, should be at agitator speed.

Measure of mixing efficiency

17. The efficiency of the mixing operation in intermingling the constituents of the concrete in all types of mixers should be such that at the end of the prescribed or permitted mixing period spread in the water-to-fines ratios, W/F, of three 1500-gram samples of mortar from well separated localities in the mixer, should be within ten per cent of the average of the three samples; W being the mixing water and F being the cement and other fine material passing the No. 100 screen.*

18. The ratio of coarse aggregate to mortar should appear to be uniform in all parts of the mixer on visual inspection and this should be borne out by subsequent observation of the workability of the concrete.

The discharge operation

19. No portion of the time required for discharging any type of mixer should be considered a part of the required net mixing time. The discharge facilities of all types of mixers should be capable of ready discharge of concrete of the stiffest consistency which should be placed by means of vibration. To meet this requirement, all mixers should be capable of ready discharge of concrete of 1 inch slump.

20. To preserve the uniformity of distribution of materials and the usual homogeneity of the concrete in the mixer immediately prior to discharge, all types of mixers discharging into hoppers, buckets, cars, etc., should be so equipped that the concrete will drop vertically, not diagonally, into such containers in accordance with the principles shown in the sketches which constitute a part of these recommendations. Separation of coarse aggregate from the mortar, commonly resulting as concrete is discharged from the mixer, is thus avoided.

21. The blade arrangement and discharge mechanism of all types of mixers, including agitating, shrink, and truck mixers, should be such that the amount of aggregate larger than $\frac{3}{4}$ inch in any portion of the batch will not differ by more than 20 per cent from the amount of such aggregate in any other portion of the batch. This may be determined readily by comparing the weights of coarse aggregate retained when at least 200 lb.† of concrete from each portion of the batch in question is washed over a $\frac{3}{4}$-in. screen. Until or unless equipment is available which will meet this requirement, and is operated in this manner, when

*For one method of test see "Mortar Test for Mixer Efficiency," Method B, U. S. Bureau of Reclamation Concrete Manual, third edition, January 1941, p. 419.

†A 500-lb. sample is preferable if it does not contain portions of the batch other than those supposedly represented.

the fraction of the batch containing excessive amounts of coarse aggregate is ordinarily the last portion discharged, that fraction should be withheld from discharge and mixed with the succeeding batch. As heretofore recommended in this case, the size of batches charged into the mixer should be reduced by this amount. In some truck mixers this type of separation may be avoided by reversing the direction of rotation for one or two revolutions prior to the final discharge.

General

22. In general, all of the foregoing recommendations should be considered where applicable in connection with the mixers of proper capacity for the job regardless of the size of the job or its classification.

IV. PLACING

Essential requirements

1. Only those methods and arrangements of equipment should be used which will reduce to a minimum the separation of coarse aggregate from the mortar at all points from the mixer to the forms. For highest quality of concrete *in place*, the placing equipment should be considered for its ability to handle mixes ideal for the particular work, from the standpoint of the concrete placeability and workability *in the forms* (preferably by means of vibration where this is applicable). Equipment should be capable of expeditious placing of concrete of the proper consistency, grading and maximum size of aggregate, at the rate most advantageous to good workmanship. Selection or approval of any part of placing equipment requiring, for efficient operation, mixes containing larger proportions of sand, cement, or water, or smaller coarse aggregates, than indicated for best results on the above basis, is not recommended.

Avoidance of separation

2. The most important consideration in handling and placing concrete is the avoidance of separation of coarse aggregate from the concrete. Particular attention must be paid to that tendency at ends of chutes and conveyor belts, at hopper gates and at all other points of discharge, if uniformity and homogeneity of concrete in place and good workmanship are to be assured. No consideration should be given to the common fallacy that separation occurring in handling will be eliminated in the course of other operations. Separation should be prevented—not corrected after its occurrence.

3. Important in arranging equipment to prevent separation is the provision that the concrete shall *drop vertically* into the center of whatever container receives it. The importance of this increases very greatly with increases in slumps, maximum size, amounts of coarse aggregate,

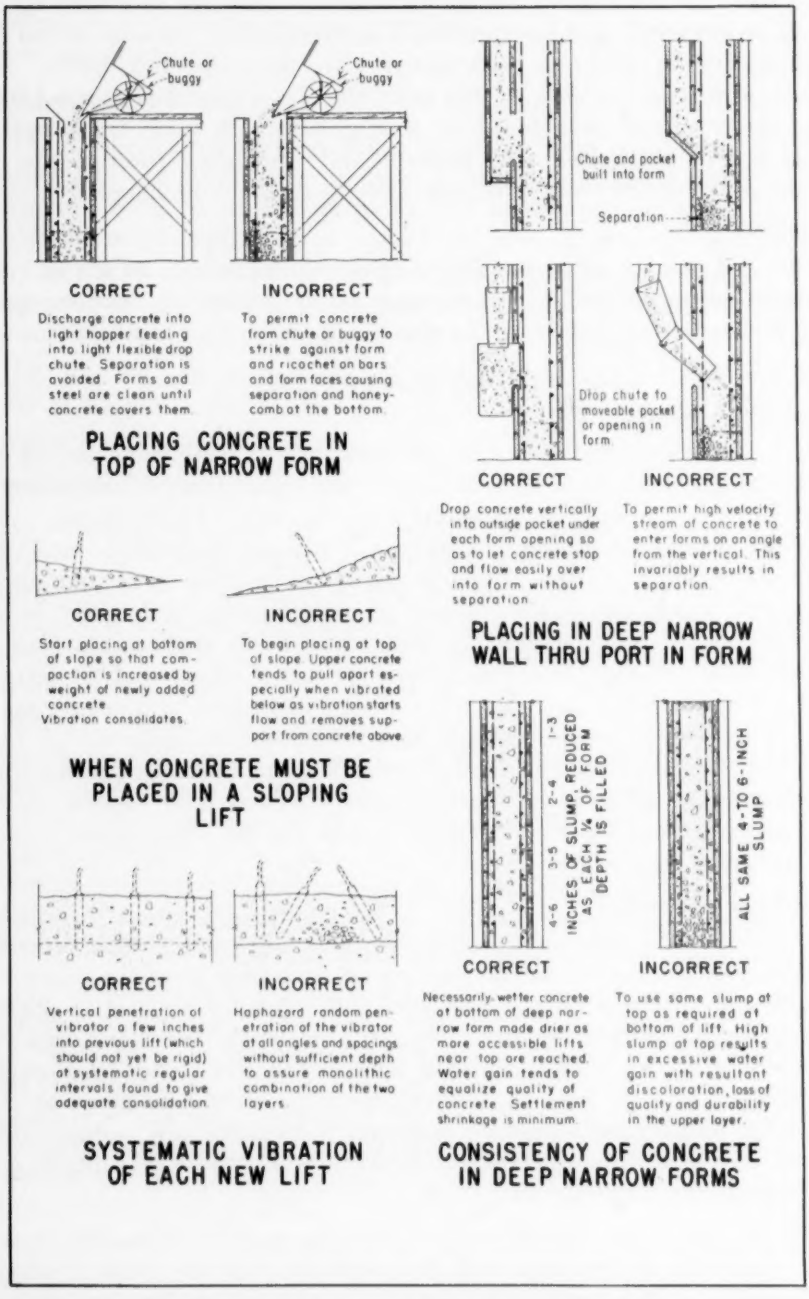
Chute or buggy
Chute or buggy
CORRECT
INCORRECT
Discharge concrete into light hopper feeding into light flexible drop chute. Separation is avoided. Forms and steel are clean until concrete covers them.
To permit concrete from chute or buggy to strike against form and ricochet on bars and form faces causing separation and honeycomb at the bottom.
PLACING CONCRETE IN TOP OF NARROW FORM
CORRECT
INCORRECT
Start placing at bottom of slope so that compaction is increased by weight of newly added concrete. Vibration consolidates.
To begin placing at top of slope. Upper concrete tends to pull apart especially when vibrated below as vibration starts flow and removes support from concrete above.
WHEN CONCRETE MUST BE PLACED IN A SLOPING LIFT
CORRECT
INCORRECT
Vertical penetration of vibrator a few inches into previous lift (which should not yet be rigid) at systematic regular intervals found to give adequate consolidation.
Haphazard random penetration of the vibrator at all angles and spacings without sufficient depth to assure monolithic combination of the two layers.
SYSTEMATIC VIBRATION OF EACH NEW LIFT
Chute and pocket built into form
Separation
Drop chute to moveable pocket or opening in form.
CORRECT
INCORRECT
Drop concrete vertically into outside pocket under each form opening so as to let concrete stop and flow easily over into form without separation.
To permit high velocity stream of concrete to enter forms on an angle from the vertical. This invariably results in separation.
PLACING IN DEEP NARROW WALL THRU PORT IN FORM
1-3
2-4
3-5
4-6
INCHES OF SLUMP, REDUCED AS EACH ¼ OF FORM DEPTH IS FILLED
ALL SAME 4-TO 6-INCH SLUMP
CORRECT
INCORRECT
Necessarily wetter concrete at bottom of deep narrow form made drier as more accessible lifts near top are reached. Water gain tends to equalize quality of concrete. Settlement shrinkage is minimum.
To use same slump at top as required at bottom of lift. High slump at top results in excessive water gain with resultant discoloration, loss of quality and durability in the upper layer.
CONSISTENCY OF CONCRETE IN DEEP NARROW FORMS

Fig. 3—Correct and incorrect methods of placing concrete

and with reductions in cement content. Falling concrete should be closely confined in a down pipe of the proper size to within a few feet of the place of deposit in the forms or other container *and the final drop must be vertical* if separation is to be prevented. Application of this principle to various common details of concrete handling equipment is shown in the accompanying sketches.

4. Where applicable, the use of bottom discharge buckets is a superior method of handling and placing mass concrete, provided: (1) they are of a size, and may be discharged in a manner and with such frequency, that the concrete may be placed in approximately horizontal layers while the previous layer is still soft; (2) they are capable of discharging concrete of the stiffest consistency specified; (3) successive batches are so placed as to afford opportunity for thoroughly working the concrete by means of internal vibrators; (4) only complete mixer batches are placed in the bucket; and (5) separation is avoided in filling the buckets (see sketch).

5. Concrete should not be dropped through reinforcement steel, or into any deep form whether reinforcement is present or not, so as to cause separation of the coarse aggregate from the mortar by repeatedly hitting rods or the sides of the form as it falls. For placing under such conditions, hoppers and, if possible, vertical ducts should be used in the forms, or other means employed so that the concrete may reach the place of final deposit without separation or coating the steel and forms with mortar that will dry out long before it can be covered with concrete. In difficult cases of deposit of this kind in deep, narrow, reinforced walls, where even narrow rectangular ducts can not be inserted, good results may be obtained by closing each third space between studs on one side of the wall for a duct through which to drop the concrete which then enters the form through holes cut in the sheathing at vertical intervals not greater than 4 ft. as the concrete rises to each opening. A pocket should be provided at the bottom of each duct successively, below each opening, so that the concrete will stop and flow easily over into the forms with minimum scattering and separation. A good internal vibrator should be in operation on each side of an opening while concrete is entering the forms.

Vibration

6. Except for a few special sections, such as thin slabs, no method of compaction and consolidation of concrete in place is superior to effective internal vibration. Vibration contributes general improvement to concrete work in the fact that superior workmanship may be obtained at lower cost, and because it permits successful routine use of a concrete

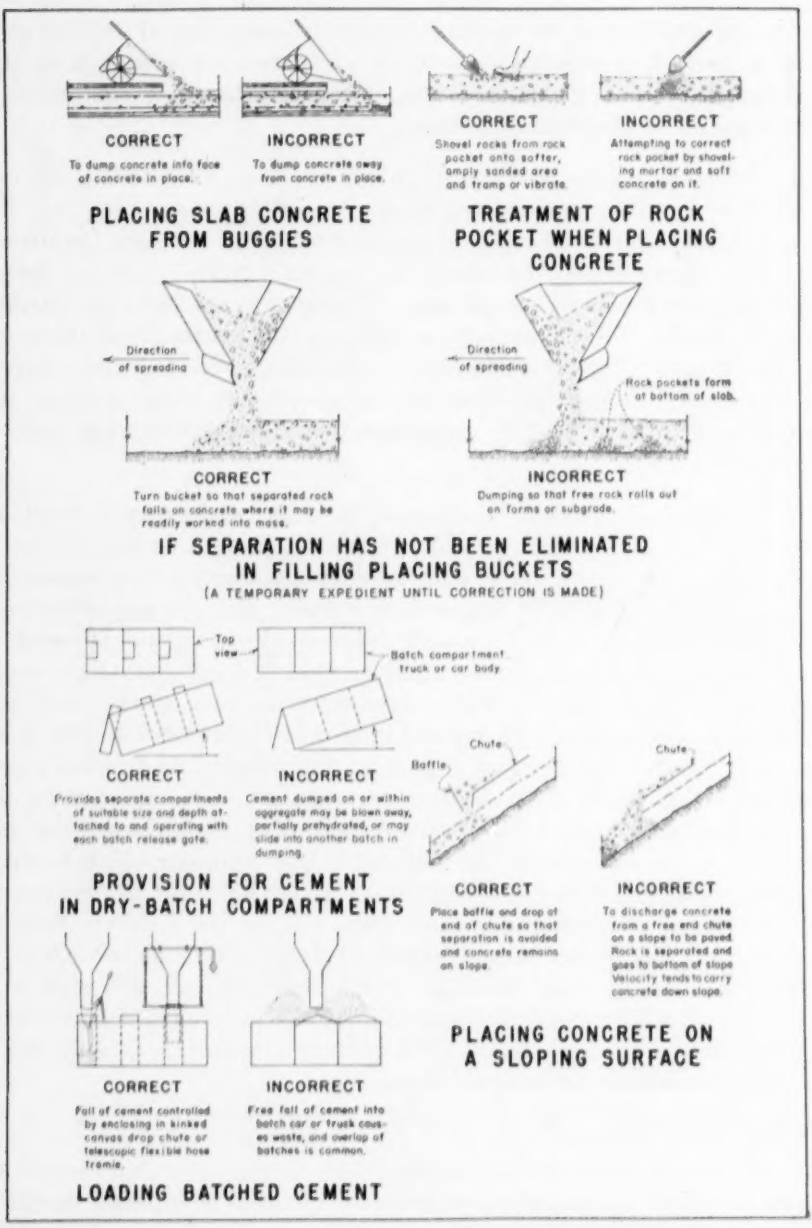
CORRECT
To dump concrete into face of concrete in place.
INCORRECT
To dump concrete away from concrete in place.
PLACING SLAB CONCRETE FROM BUGGIES
CORRECT
Shovel rocks from rock pocket onto softer, amply sanded area and tramp or vibrate.
INCORRECT
Attempting to correct rock pocket by shoveling mortar and soft concrete on it.
TREATMENT OF ROCK POCKET WHEN PLACING CONCRETE
Direction of spreading
CORRECT
Turn bucket so that separated rock falls on concrete where it may be readily worked into mass.
Direction of spreading
Rock pockets form at bottom of slab.
INCORRECT
Dumping so that free rock rolls out on forms or subgrade.
IF SEPARATION HAS NOT BEEN ELIMINATED IN FILLING PLACING BUCKETS
(A TEMPORARY EXPEDIENT UNTIL CORRECTION IS MADE)
Top view
Batch compartment truck or car body
CORRECT
Provides separate compartments of suitable size and depth attached to and operating with each batch release gate.
INCORRECT
Cement dumped on or within aggregate may be blown away, partially prehydrated, or may slide into another batch in dumping.
PROVISION FOR CEMENT IN DRY-BATCH COMPARTMENTS
CORRECT
Fall of cement controlled by enclosing in kinked canvas drop chute or telescopic flexible hose tremie.
INCORRECT
Free fall of cement into batch car or truck causes waste, and overlap of batches is common.
LOADING BATCHED CEMENT
Chute
Baffle
CORRECT
Place baffle and drop at end of chute so that separation is avoided and concrete remains on slope.
Chute
INCORRECT
To discharge concrete from a free end chute on a slope to be paved. Rock is separated and goes to bottom of slope. Velocity tends to carry concrete down slope.
PLACING CONCRETE ON A SLOPING SURFACE

Fig. 4—Correct and incorrect methods in handling materials and placing concrete

less wet than the usual "medium" consistency, resulting in more durable, higher quality concrete. The advantages and importance of vibration where applicable are now so well established that it is worthy of serious favorable consideration for small jobs of a few hundred yards, and its requirement is recommended for all larger concrete work.

7. Detailed recommendations are omitted here and the reader is referred to "Recommendations for Placing Concrete by Vibration" reported by ACI Committee 609, ACI JOURNAL, Mar.-Apr. 1936; *Proceedings* V. 32, p. 445. Sufficient to say that the name of vibration is not enough. Equipment should be powerful, of high frequency, efficient, and rugged. Operators should be experienced, competent, dependable and energetic. Ample standby units and parts should be provided. Care should be exercised that concrete is not over-vibrated, particularly if it exceeds 4 in. in slump. Vibrators should not be used improperly to "transport" concrete in the forms. Vibrators should be inserted and withdrawn at many points, from 18 in. to 30 in. apart, for short periods, (usually from 5 to 15 seconds is sufficient) in preference to insertion for longer periods at wider intervals. Systematic spacing of insertions of the vibrator should be established to insure that no concrete remains unvibrated.

8. Inadvertent or intended revibration of concrete or steel embedded in it, any time before the concrete becomes so far set that it will not again become plastic by continued vibration, is not detrimental but may actually increase the strength of the concrete and its bond with the steel. As far as the ordinary job is concerned there is little likelihood of damage from revibration of lower lifts or from vibration transmitted by the steel, provided there is avoidance of actual rupture of newly hardened concrete that will not again become plastic by the vibration. This condition can be determined by an experienced operator from the action and sound of the vibrator.

9. In some cases, vibration may be advantageously supplemented by spading at the face of the forms. The use of flattened spading tools or rods (together with concrete placed in shallow lifts) is helpful in liberating air bubbles which often deface vertical walls and invariably are found on surfaces molded by forms which slope toward the concrete. Under sloping forms vibration without spading tends to aggravate the characteristic imperfections usually found on such surfaces and overvibration is particularly undesirable. (It is in the improvement of such surfaces that absorptive form lining is finding a very valuable application.) At corners, obstructions, and other points where perfect placing may be in question, supplementary hand rodding of the concrete is desirable.

Provision for handling concrete of the proper consistency

10. Objection is frequently made on the job to a relative stiff consistency which will not flow down a certain chute, drop out of a certain hopper or discharge through certain gates, although it is freely admitted that it is readily workable in the forms, particularly when properly vibrated. Such an objection is not valid and should not be sustained if the drier consistency has been determined in advance to be of practicable workability in the forms and has been made a requirement of the specifications. It is the function of concrete handling and placing equipment to handle and place concrete of proportions and consistencies that can be properly worked in the forms and selected for the results required in the forms. Limitations on consistencies and proportions should not be imposed by inadequate chutes, hoppers or gates.

11. Thus it is important, in the design and approval of concrete handling equipment that chutes, where necessary, be amply steep, metal or metal lined, round bottomed, of large size, rigid, and protected from overflow. Discharge gates and hoppers should be large enough to pass quickly and freely concrete of the stiffest consistency likely to be found practicable for placing in the forms by means of vibration. In many cases chutes steeper than 2 to 1 should be used, and double or triple the usual area of hopper outlets and gates would not be excessive.

12. Except where loss of slump due to delayed placing is unimportant and no increase in the original amount of mixing water is made because of it, or when remixing without additional water after a period of delay is desirable and practicable for reduction of shrinkage due to setting, every effort should be made to keep as short as possible the time elapsed from the moment water and cement come together, and the arrival of the concrete in the forms. This facilitates control of uniformity of consistency of the concrete in the forms, and reduces to a minimum the water content of the concrete, and the variable loss of slump between the mixer and the forms. For this same reason concrete should not be exposed in thin streams in long chutes or on long conveyor belts in which condition it is subject to the variable effects of all kinds of weather. It should not be necessary to mix the concrete any wetter than it may be worked in the forms by means of vibration.

V. GENERAL CONSIDERATIONS

1. In mixing and placing concrete, all concerned should remain aware that shrinkage cracks and lack of durability are primarily proportional to the volume of mixing water per unit volume of concrete within the range of practical and suitable mixes for most work. Though in many

cases they are, they *may* not be correlative to the water-cement ratio except for mixes otherwise identical in aggregate and cement content, or for mixes identical only in water content and consistency. An example is the case in which both cement and water are added without changing the water-cement ratio but with an increase in slump. Another, when increases in cement and water are accompanied by use of finer grading of aggregate resulting in little change in consistency. Such variations in unit water content proportionately affect the ultimate quality of the concrete without relation to the water-cement ratio. The more a form is filled with the right combination of solids, and the less it is filled with water, the better will be the resulting concrete. Thus to this end, moderation in the use of water, cement, and fine aggregate, together with the use of aggregate graded to the largest practical maximum size, all in conformity with the elemental needs of the work, should be consistently practiced. Only as much cement should be used as is required to obtain adequate strength and other essential properties; and only sufficient water and fine aggregate should be used as is required to obtain concrete with no more than a degree of workability which may be properly worked in the forms by means of vibration or by more appropriate methods where vibration is not applicable.

2. Sufficient mixing and placing capacity (with judgment in planning the progress of the work) should be provided so that the work may be kept alive and free from "cold joints." In all formed structures, including tunnel linings, concrete should be placed in horizontal layers of not greater depth than 2 feet, particularly avoiding inclined layers and inclined construction joints. It is very important that each layer be placed while the previous layer is still soft. Concrete should not be allowed or caused to flow horizontally or on slopes in the forms. Where concrete must be placed on a slight slope, placing should begin at the lower end of the slope and progress upward thereby increasing the compaction of the concrete. Pneumatic means of placing concrete should, in general, be avoided wherever an alternative method may be used.

Construction joints

3. For the sake of appearance, irregular construction joints should not be permitted. Each lift of concrete should be filled up to a temporary grade strip, or preferably a "V" or a slightly beveled rectangular strip should be left on the forms at the line of the joint where such grooves can be located so as not to detract from the appearance of the work and so as to be less unsightly than the usual construction joint. Particular precautions should be taken to secure the forms tightly against the concrete at the joint, since, otherwise, an unsightly offset will occur

and mortar from the subsequently placed concrete will disfigure the surface of the concrete in place.

4. The surface of the construction joint should be prepared in a manner that will insure bonding with the concrete later placed on it, and if the joint must be watertight the preparation of the joint must be particularly thorough. It is not difficult to obtain a good joint where good quality low-slump concrete has been used at the top of the lift and has not been overworked. Where wet consistencies have been used, or where excessive vibrating or working has brought water or mortar to the surface, the concrete at the surface of the lift is usually so inferior that it is not easy to obtain a good joint.

5. In the latter case the so-called initial jet cleanup is a good expedient for such an inferior joint condition, which on good work would not be encountered. The first step should be the removal of all laitance and inferior surface concrete and the washing of mortar from protruding aggregate by means of a strong jet of air and water, at approximately 100 psi pressure, after the concrete has hardened sufficiently to prevent the jet from raveling the surface below the desired depth and from forming cloudy pools of water that will leave a film on the surface when they dry. The surface of joints so treated should be especially well moist-cured, preferably by a 1-inch layer of saturated sand, and, if possible the surfaces should never be permitted to dry out during the interval before concrete is placed on it. Before placing the new concrete the surface should be restored to the bright clean condition existing immediately following the initial jetting by means of vigorous brushing with fine wire hand brushes or by sandblasting as necessary. If the surface has been properly wet sand cured, very little final scrubbing or sandblasting will be necessary.

6. Initial jetting, followed by a thorough final cleanup can also be used with good results where quality of concrete at the joint surface is good. It produces no better results, however, than can be readily obtained, on surfaces which have been properly placed at the right consistency, by omitting any initial treatment and removing the surface film and dirt to a bright new surface by sandblasting and washing immediately prior to placing the new concrete. The initial jetting method including a comparable final cleanup is usually found to be more expensive than the final sandblasting method and is not as foolproof. Final sandblasting without initial cleanup has been found to produce excellent results economically on horizontal joint surfaces of mass concrete that has been placed at dry consistencies with the aid of vibrators, if the surface is protected from excess working due to

keyway construction, the endeavor to embed all coarse aggregate, and the general job traffic until the concrete has hardened.

7. In all cases the new concrete should be preceded by about one-half inch of soft mortar of the same proportions as that in the concrete. When accessible, this should be scrubbed into the surface of the joint with wire brooms. In column forms and deep narrow forms where one-half inch of mortar may seem inadequate, it is preferable to follow the mortar with several inches of concrete containing mortar in excess of that in the usual mix and possibly containing coarse aggregate somewhat reduced in maximum size rather than use large quantities of straight mortar.

Finishing of unformed surfaces

8. For the most durable results in any type of finished, unformed concrete surfaces, the following general requirements should be made. Concrete should be used of the stiffest consistency that can be properly placed and worked or vibrated in accordance with the finishing process adopted. In the initial operations of screeding, floating, and first troweling, the surface of the concrete should be worked *as little as possible* in obtaining the desired result. Each step in the finishing operation from the secondary floating to the final operation should be delayed *as long as possible* and yet permit the desired result to be obtained. The use of any finishing tool in any area where water has accumulated should be prohibited. Dry topping and mortar topping should be avoided. The surface of the concrete should be directly finished to the texture desired.

Cold weather concreting

9. For concrete in sections thinner than would be called mass concrete, when the weather is such that freezing of the concrete may occur, arrangements should be made to heat the mixing water and the aggregate so that the temperature of the fresh concrete in place is approximately 70F. At this temperature setting commences at once and, due to the heat liberated by hydration, will progress at a normal rate, provided the work is protected for several days from dissipation of heat more rapidly than additional heat is generated. This procedure is eminently preferable to that of placing the concrete cold at any temperature above freezing and then endeavoring to heat it in place by salamanders or other means; however, under severe conditions it may be necessary to surround the work with some form of heating protection in addition to protection by insulation. Steam released under a tarpaulin enclosure is an excellent protection since a moist atmosphere favorable to curing as well as protection from freezing is afforded. Corners, edges and surfaces of concrete are particularly vulnerable to

freezing and the need for their adequate protection cannot be too strongly emphasized. In fact they may be frosted when the dry bulb temperature is above 32 F. if the wet bulb temperature reaches 32 F. In connection with cold weather protection, such precautions should be taken that the curing of the concrete will not be impaired and that no portions of the work will become overheated. It is important that there should be a curing period of sufficient length at temperatures above freezing so that when it is exposed to temperatures below freezing at the end of the curing period the concrete will not be injured.

10. With adequate protection of the surfaces from freezing, the minimum temperature of freshly mixed mass concrete may be permitted to be as low as 40F. when placed, because the heat of hydration is lost much more slowly from this type of concrete.

Hot weather concreting

11. For best ultimate quality, concrete should be placed at the lowest practicable temperature during hot weather. Effective results can usually be accomplished by concreting only at night, sprinkling or cooling the aggregate, avoiding hot cement, and using very cold mixing water. Any combination or degree of these practices that may be feasible for the particular work are important and should be required. Curing should preferably be obtained by sprinkling or covering with moist burlap for its additional cooling value. If curing must be done by means of black bituminous sealing compounds, they should be given a coat of whitewash promptly so as not to expose the heat absorbing black surface to the sun.

Forms

12. Provisions as to certain types of forms are necessary because, when they are not followed, concrete containing excessive amounts of sand, cement, or water is usually necessary in order to obtain acceptable workmanship. Horizontally moving slip forms should not be used on slab work on which standard types of paving and finishing machines can be used. Preferably such machines should be equipped with an effective form of vibration or mechanical device for working and compacting the concrete. A continually moving slip form, however, should be used in preference to fixed forms whenever possible on slope paving slabs or steeply curved inverts.

13. Workmanship in placing concrete is largely judged by the appearance of the work on removal of the forms, and appearance in itself is an important quality which must usually be obtained when the concrete is placed. It is therefore necessary in these recommendations to mention other requirements for satisfactory results.

14. With the general adoption of vibration of concrete, forms must not only be built substantially, but they must be tight, since, otherwise, unsightly sand streaks and gravel pockets will be caused by loss of mortar made unusually liquid during vibration. Exceptional care should be exercised to insure that all form panel joints, corners, and connections and all seams between all types of sheathing are completely tight immediately prior to placing the concrete. Because of their superior tightness where they penetrate the forms, tie rods are recommended in preference to tie wires. Where wires must be used holes for them should be as small as possible to keep mortar leakage at a minimum.

15. Forms should be protected from deterioration, weather and shrinkage prior to concreting by proper oiling or by effective wetting. Form surfaces should be clean and of uniform texture. Where they are permissible, re-used forms should be carefully cleaned and oiled. Steel forms should be thoroughly cleaned but never sand blasted nor abraded to bright metal. Where "peeling" is encountered with steel forms, leaving the cleaned, oiled, forms in the sun for a day, or vigorously rubbing the affected areas with liquid paraffin will usually improve the condition. It is generally less expensive and more satisfactory to obtain the desired surface effect by proper treatment and preparation of the forms than it is to obtain it by working over the concrete after the forms are removed.

VI. CLOSURE

1. In general, all of the foregoing features should be considered, insofar as applicable, in connection with each job classification, regardless of type or capacity of equipment.

This report will be open for discussion at the 38th Annual Convention on a proposed motion by the Committee for the adoption of the report as an ACI Standard. Should the motion prevail by a majority vote of members present the report will be submitted by reference to letter ballot of ACI membership. "An amendment to the proposed Standard presented on the floor of the convention shall be sustained only when approved by nine-tenths of members voting." All discussion intended to influence convention action should therefore be available to the ACI Secretary in triplicate by Feb. 1, 1942 unless it is to have convention presentation by its author. Discussion is scheduled for publication in the JOURNAL for June 1942.

A part of PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE Vol. 38

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Vol. 13 No. 6 7400 SECOND BOULEVARD, DETROIT, MICHIGAN June 1942

Discussion of Report of Committee 614:

Proposed Recommended Practice for Measuring, Mixing and Placing Concrete*

CONVENTION DISCUSSION BY LEWIS H. TUTHILL, C. H. FORESMAN

By LEWIS H. TUTHILL, CHAIRMAN OF COMMITTEE 614
(in presenting report for Convention adoption)

To give you an idea of the consideration given to the preparation of the report of Committee 614, I will outline very briefly the steps in its development.

Five years ago in March 1937, Committee members were requested to discuss a list of pertinent subjects and any others they considered appropriate. These discussions formed the basis of the first draft of the report which was distributed for comment. Repeating this process, a third draft was ready for discussion at a meeting held during the 1938 convention.

There was considerable interest in evidence at this meeting, which 20 interested persons attended, including about half the committee members, and the meeting lasted about 8 hours. From the discussion a fourth draft was shortly prepared which was approved by a good majority of the Committee, the others being satisfied with the report except for certain implications which might be construed to the disadvantage of truck mixing on engineering construction outside of metropolitan areas. The report was then submitted to the Standards Committee, with a few further revisions suggested by members of Committee 614.

The Standards Committee did some very beneficial editing of the report and returned it to us in November 1939. After a few further revisions as a result of comment received in the meantime, the report was again referred to a vote of the Committee and approved by a satis-

*ACI JOURNAL, Nov. 1941; *Proceedings* V. 38, p. 93.

factory majority. Transit mixers were still causing lack of unanimity but it was agreed that the report might be published for the information of the Institute. This was done and the report appeared in the JOURNAL for February 1940. The September JOURNAL of that year carried subsequent written discussion together with Committee approved closure and amendments formulated from discussion and comment received following publication of the report. Amended in this form, the report was again submitted to the Standards Committee which suggested some further revisions and further effort toward unanimity. At a meeting in Washington at the 1941 convention common ground was found on all points of previous difference. During 1941, these and a later group of amendments were submitted to Committee 614 and all but unanimous approval was at last secured, one or two members not voting. The Standards Committee approved the report for presentation to this convention on a motion for adoption as an ACI Standard—as published in November 1941.

The report is the result of the efforts and good advice of many men; as Chairman of Committee 614, I want to express my appreciation for this assistance and cooperation from members of Committee 614 and many others.

The Committee is not coming before you with the idea that the report is the last word on the subject because every forward-looking individual realizes that each new standard is merely a point of departure for a better one. However, every effort has been made to give you a mature and carefully considered recommendation. The report is now before you and I move it be adopted as a Standard of the Institute.

By C. H. FORESMAN*
(read by title)

I have never seen a more comprehensive exposition of the principles of the manufacture of good concrete and of proper concrete construction than is contained in the report of Committee 614. Any legitimate criticism could be as it refers to minor detail only and I submit the following in this spirit.

Part I, Paragraph 2. A specification should not deal in "Common Practice" unless common practice is also "Best Practice" or is absolutely demanded by practicability. The reference to specifications is not necessary and because of the implication that specified procedures should or could be less than the best, I suggest that the sentence containing this reference be made to read—"It is believed, however, that the interests of progress . . . etc."

*Concrete Engineer, The Board of Public Education, Pittsburgh, Pa.

Part II, Paragraph 2. I believe the separation of coarse aggregates into sizes with more narrow limits should receive more emphasis. In concrete for general construction, uniformity of workability is extremely important. Much of the variation in workability comes from segregation of coarse aggregates in transportation, storage and handling and the reduction of the range of sizes materially reduces such segregation. The Committee has covered the point here but it seems to me that a little more stress would be in order.

Part II, Paragraph 4. It is stated here that sand passing a ⅛-in. sieve should contain 20 per cent finer than 50-mesh. The subject of fine material cannot be discussed properly in one or two sentences and as this is stated it is apt to be misleading. The percentage of fines is a function of the combined aggregates and not of the sand alone. It depends, among other things, upon the maximum size of the combined aggregates and the conditions surrounding the placing of the concrete. It does not seem to be in order to state here a desirable percentage of fines for any particular sand. The paragraph is ample without that sentence.

Fig. 3 shows diagramatically the effect of water gain in deep, narrow forms and gives one method for offsetting such effect. The control of water gain is not discussed. The method shown may work under some conditions but in many cases it does not work. In any case there is a distinct uncertainty involved and any uncertainty is a detriment. It is better to have a less high quality material of known characteristics than one that is uncertain, regardless of how high the average quality may be. I would suggest that the control of water gain be discussed in connection with any method of offsetting its bad effects.

As I have said before, the points mentioned above are of minor importance and criticism of them does not imply criticism of the work of the committee. If the report is given the prominence and publicity it deserves, it should do much to improve the quality of concrete and concrete work.

(Mr. Tuthill's motion seconded, the report was adopted by the Convention as an Institute Standard, without dissent and referred to letter ballot*)

CLOSURE BY MR. TUTHILL

Probably the phrase "and not a specification" could well be omitted from Part 1, paragraph 2. It was included in an effort to satisfy critics who thought that some of the suggestions in the report were so advanced

*Canvassed May 25, 1942, with ratification of the Convention adoption of the report as an ACI Standard (ACI 614-42).

that difficulties would arise if they should be copied indiscriminately into specifications for some types of work. Paragraph 3 suggests that deviation from the recommendations may be necessary in some instances and the reader must make his own decision as to how far it is profitable for him to go.

Separation of aggregates into sizes with more narrow limits is indeed important in securing uniformity of workability and of concrete quality generally. The committee went as far as it dared in promulgating the recommended limits for the ratio of maximum and minimum sized aggregate. On the other hand, it has been pointed out to the Committee already that such a common size as No. 4 to ¾-in. aggregate is unacceptable on the basis of the Committee's recommendation that the ratio of maximum to minimum size of aggregates smaller than 1-in. not exceed 3. This ratio for No. 4 to ¾-in. aggregate is 4. It would be better to divide this material at ⅜-in. into two sizes each with a ratio of 2. At least the maximum size of No. 4 minimum material should not be greater than ½-in. or at most ⅝-in. Although No. 4 to ¾-in. material is a common size, it is also a size commonly causing trouble from lack of uniformity due to segregation within its wide range of sizes. The Committee does not disagree with Mr. Foresman on this point.

Regarding the matter of fines in sand, it is realized that this as well as many other topics within the report could be considerably amplified but an effort was made to keep the text down to fairly concise statements of the most pertinent factors. It is believed the last sentence in Part II, paragraph 4, should prevent the statement on fines from being seriously misleading.

Space limitations also apply in connection with discussion of the figures included in the report. It was believed that these four pages would add more to the value of the report than anything the Committee could write on four additional pages of text. It is true the figure mentioned does suggest discussion of the control of bleeding and water gain. Actually this subject has been ably and fully discussed in ACI literature and as far as job practice goes it boils down to more fines and less water. With the exception of adding cement for this purpose, these expedients have been discussed and optimum fines and minimum practicable slumps have been recommended.

Mr. Foresman's thoughtful and encouraging discussion is appreciated by the Committee. We hope, as he suggests, that the report will do much to improve the quality of concrete and concrete work.

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A Vacuum Method of Measuring the Air Content of Fresh Concrete*

By GEORGE L. PIGMAN†

SYNOPSIS

A method of measuring the air content of freshly mixed concrete has been developed, in which the air is removed at the reduced absolute pressure of approximately 17 mm of mercury.

The determination of the air content is made entirely by volume measurements and does not require any information on the specific gravity of the components of the concrete.

The reproducibility of results was satisfactory and varied with the wetness of the mix, being better for the wetter concretes with a flow of 85 to 110 than for concrete with a flow of 55. The air content was increased by use of a finer sand, but was reduced by increasing the cement content of the mix.

It was found that concretes normally containing less than one per cent of air had the air contents increased to values ranging from 1.5 to 14 per cent by the addition of 0.05 per cent of various aerating agents.

I. INTRODUCTION

This paper describes a method of determining the air content of fresh concrete, the method consisting essentially of measuring the amount of water required to replace the air removed from a given volume of concrete during agitation under reduced pressure. This method does not require specific-gravity determinations which are necessary in computing the air content from volume measurements, and by use of vacuum effects a more complete removal of the air, especially from concretes containing air-entraining agents, than has been accomplished by the previous method of Pearson.(‡)

Although the distribution of air is also of importance, the measurement of the total volume of air only is here considered.

*Received by the Institute June 27, 1941.
†Junior Materials Engineer, National Bureau of Standards.
‡J. C. Pearson and H. G. Collins, ACI JOURNAL, Jan.-Feb., 1936 *Proceedings* V. 32, p. 235.

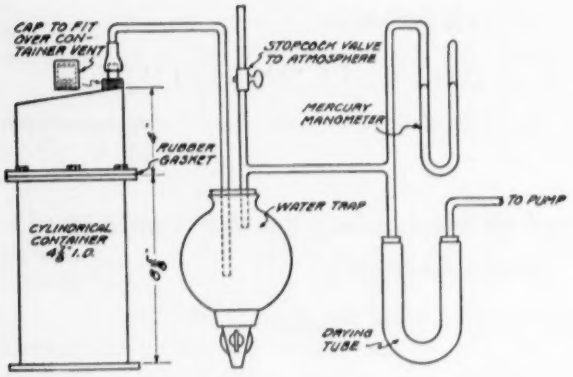

Fig. 1—Schematic diagram of apparatus

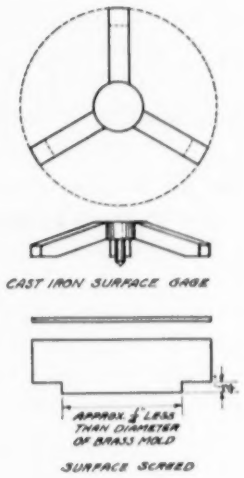

Fig. 2 (at right)—Surface gage and screed

II. SCOPE

The following tests were made:

1. The air was extracted from a 1:2:4 mix at various pressures from atmospheric (750 mm) to 17 mm of mercury. For all other tests made, the air was extracted at a pressure of approximately 17 mm of mercury.

2. Determinations were made of the amounts of air incorporated into two mixes, a 1:4.5 and a 1:6, respectively, parts of cement to parts of aggregate by weight, in which each of ten aerating agents had been introduced by grinding with the cements.

3. A few determinations were made of air incorporated in mixes with sands of two finenesses; and of mixes containing various admixtures. The amount of air incorporated when the mixes were compacted into molds by standard rodding and by vibration was also determined.

4. The air content was determined on each of six mixes. One part of cement was used to 5.67 parts by weight of aggregate, the ratio of sand to coarse aggregate being varied, as well as the water content, to give several consistencies. Ten measurements were made on each mix to furnish information on the reproducibility of the test.

III. APPARATUS

A schematic diagram of the apparatus is shown in Fig. 1. The container in which the concrete is placed is a brass cylinder of 4⅞ in. inside diameter, divided into two sections. The lower section is 8⅝ in. deep. The upper section, or cover, is 4 in. high and is bolted to the lower section

through flanges, with a 1/16-in. rubber gasket in the joint. Protruding one inch from the top of the cover is a 1/2-in. brass tube, providing a connection for a rubber suction tube. The cap is also provided for sealing this connection. The height to which the lower section is filled is determined by a gage (Fig. 2). The correct height is indicated when the gage point will just break the surface of a water layer on the concrete as the gage is carefully lowered into position.

The volume of the lower section, below the level of the gage point, was obtained by measuring the volume of the water required to fill it to the gage-point level. The measurement was made by means of a 1-liter volumetric flask and a 100-ml burette. The average of 6 determinations gave a value of 2555 ml. The volume above the gage point, after the cover had been bolted in place, was determined by the amount of water added to bring the level to the top of the 1/2-in. tube. The average of 6 determinations was 1042 ± 0.5 ml.

A glass separatory funnel (Fig. 1) is used as a water trap, and a drying tube, containing about 75 g of magnesium perchlorate trihydrate is placed in the suction line between the trap and the vacuum pump. A connection is also made to a mercury manometer.

IV. TEST PROCEDURE

The tests were made in a laboratory maintained at 70 ± 2 F., in which the dry materials and water were stored long enough, prior to the test, to reach laboratory temperature.

A sample of approximately 8 kg is required. The concrete is mixed for 2 minutes, permitted to stand for 5 minutes and then rodded or vibrated into the lower section. The top surface of the concrete is then levelled and brought to a definite height (about 1/4 in. below the top edge of the lower section) by means of a special screeding or leveling tool (Fig. 2). The screed is drawn over the surface to remove excess material, and then a small amount (about 2 ml) of water is added to form a level liquid surface which is brought to the height of the gage point.

The cover is next bolted to the lower section, a measured 100.0 ml of water added and the cap screwed on the opening. The contaner is then rolled vigorously on a table for 5 minutes to mix the water and concrete thoroughly and to allow the main portion of the entrapped air to be freed. After the suction tube and trap have been wetted with water and allowed to drain, the suction line is attached to the container, the pressure lowered to a predetermined value, indicated on the manometer, and the rolling process continued for an additional 8 to 10 minutes.

After release of the vacuum, the water carried into the trap during the suction period is drained back into the container. The trap and

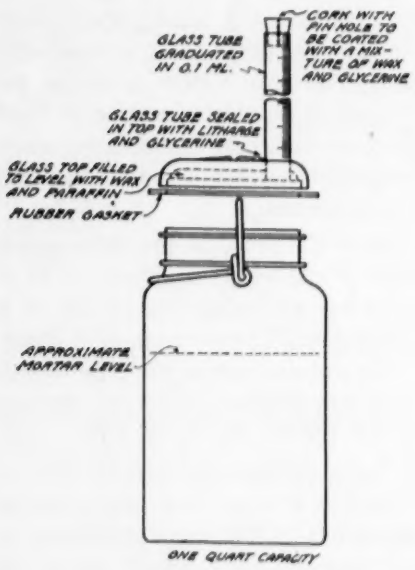
CORK WITH PIN HOLE TO BE COATED WITH A MIXTURE OF WAX AND GLYCERINE
GLASS TUBE GRADUATED IN 0.1 ML.
GLASS TUBE SEALED IN TOP WITH LITHARGE AND GLYCERINE
GLASS TOP FILLED TO LEVEL WITH WAX AND PARAFFIN
RUBBER GASKET
APPROXIMATE MORTAR LEVEL
ONE QUART CAPACITY

Fig. 3—Jar for shrinkage test

connecting tube are then cleaned by flushing with one or more measured portions of water, this water also being drained into the container.

The container is finally filled to the top of the ½-in. tube with water from the burette, and the total volume of added water (Vw) is recorded. During addition of this water the container is tilted and shaken to allow air bubbles to escape and from one to ten drops of amyl alcohol are added to the surface to disperse the foam.

The drying tube is weighed before and after the test, and the gain in weight in grams (ΔW) is obtained.

A simple auxiliary test was used in order to correct for the decrease in volume of the sample which occurs between the time the surface of the concrete is levelled to the gage point and the time of the final filling of the container with water. In this auxiliary test a weighed one-quart glass jar (Fig. 3) is almost completely filled with concrete immediately after the mixing period. The weight of the filled jar is obtained, and the glass cover with open graduated tube is attached. The jar and a portion of the graduated tube are then filled with water and shaken to separate the major portion of the entrapped air and foam. Readings are then made of the water level at the time the concrete in the lower section is leveled to the gage point, and again as the lower section is filled with water in the final volume determination.

The change in volume, ΔV_c, of the concrete in the test cylinder which occurs in normal setting is calculated as the decrease in volume of the

concrete in the auxiliary jar, multiplied by the ratio of the weight of the test sample to the weight of the auxiliary sample.

The volume of air, V, released from the concrete is calculated by the equation

$$V = V_w - 1042 - \Delta W - \Delta V_c$$

in which V_w is the sum of the various portions of water added as diluent, flushing agent, and filling agent, ΔW is the volume of water absorbed in the drying tube (determined from the change in weight). The corresponding percentage value of the air released is $100\ V\ /\ 2555$.

V. TEST DATA

1. Results obtained at various reduced pressures

A series of tests was made to determine the effect of differences in the suction used in the procedure upon the measured air content of a fairly wet (1:2:4) concrete (flow = 80). The results are shown in Table 1.

TABLE 1—THE AMOUNT OF AIR EXTRACTED AT DIFFERENT REDUCED PRESSURES

Absolute Pressure mm. of Mercury	Percent Air Extracted
750*	0.91
105	1.13
33	1.20
30	1.16
19	1.32
17	1.27

*Atmospheric pressure.

The table shows that the air removed from the mix by making the test at atmospheric pressure was 0.91 percent of the original volume of concrete. The tests made at various reduced pressures which ranged from 17 to 105 mm of mercury, showed that 1.13 to 1.32 per cent of air was removed, all substantial increases over the amount removed at atmospheric pressure. There was no appreciable difference in the amount removed at 30 to 105 mm absolute pressure, but at a pressure of 17 to 19 mm a slightly greater amount was removed. As the vapor pressure of water at 70F is 18.6 mm, which vapor pressure is slightly lowered by dissolved salts and alkalies from the concrete, the highest vacuum shown in the table is the limit for the test, inasmuch as a greater vacuum would boil off the water from the concrete to such an extent as to exceed the absorption capacity of the drying tube.

It is apparent from these data that the process of rolling concrete in a closed container at atmospheric pressure with an excess of water does not free all the air from the concrete. All further tests therefore were made under a pressure of approximately 17 mm of mercury.

2. Air content of various mixes

The test procedure was applied to concretes with cements which contained aerating agents added as grinding aids. Table 2 gives the results obtained in these tests.

TABLE 2—EFFECT OF VARIOUS ADDITIONS ON THE AIR CONTENT OF CONCRETES

Additions*	1:2.4:3.6 mix		1:1.8:2.7 mix	
	Air Content		Air Content	
	Measured Percent	Computed Percent	Measured Percent	Computed Percent
Ammonium stearate	2.17	2.38	1.64	1.48
Sulfonated petroleum	2.48	2.50	2.57	2.38
Sulfonated tallow	5.77	5.55	5.43	5.51
Fish oil stearate	7.23	8.77	7.50	7.32
Tallow	9.24	9.08	8.57	8.70
Vinsol resin	.....	.....	11.2	12.7
Wetting agent	13.2	.....	10.8	.....
Waste sulfite liquor	14.3	14.3	13.1	12.9

*0.05% (by weight of the cement) of each addition was added to the clinker before grinding.

Two concrete mixes were used in this series, namely, 1:2.4:3.6 and 1:1.8:2.7 by weight and with water contents adjusted to give flows* of 50 to 55. 0.05 per cent of each aerating agent had been added to respective portions of the same clinker before grinding.

As shown in Table 2, the air content varied widely with the aerating agent, but was not greatly affected by the change in mix.

This particular series of experiments was a severe test of the method of procedure adopted, since an increase in the measured air content was accompanied by an increase in the amount of foam appearing on the concrete as a result of agitation during rolling. In most cases the free space in the metal container was entirely filled with foam at the end of the initial rolling period at atmospheric pressure. This foam was particularly stable and difficult to disperse, yet contained sufficient volume of water so that it could not be discarded.

It was found that the foam could be broken up by drawing off a sufficient quantity of it to fill about $\frac{1}{5}$ of the trap. After the connection from the metal container was closed, further suction caused the foam bubbles in the trap to expand and break. By a continued repetition of this process practically all of the foam produced could eventually be broken up. The slight amount of scum and foam remaining after this process could usually be dispersed by the addition of a drop of amyl alcohol.

*The flow was determined on concrete stirred into the mold, and recorded as the increase in diameter after fifteen ¼-in. drops in 10 seconds on the 30-in. flow table. This method is not quite the same as the A.S.T.M. procedure (Proc. A.S.T.M. Flow Table Test for Portland-Cement Concrete, C-124-39).

Another group of tests was made, using a 1:2.67:3.00 mix, data on which are not presented in the tables but which can be summarized as follows: (1) Compaction of the concrete by vibration resulted in a slight decrease of the air content. Typical example: A concrete with a flow of 60 had by normal rodding procedure 1.11 per cent air. Upon vibration of a similar concrete the air content of 0.82 per cent was obtained. (2) The addition of fly ash in amounts up to 15 per cent did not appreciably affect the air content, but the addition of 2.0 per cent $CaCl_2$ increased the air content from 0.77 per cent up to 1.48 per cent. The concretes containing these admixtures had a flow of 110. (3) The use of a coarser sand decreased the air content of the concrete, especially in the drier concretes. Thus changing the fineness modulus of the sand from 2.35 to 3.10 decreased the air content as follows: At a flow of 110, 1.43 to .77 per cent air; at a flow of 60, 1.76 to 1.11 per cent air; at a flow of 15, 4.12 to 1.78 per cent air.

3. Tests of mixes of several gradings and of several consistencies: reproducibility of results

In another series, concrete mixes representing three sand-gravel ratios and three cement-water ratios were used. The mixes were all 1:5.67 by weight. Ten tests were made in each group, in order to furnish information on the reproducibility of the test. The results are given in Table 3.

TABLE 3—REPRODUCIBILITY OF AIR-CONTENT DETERMINATIONS ON TYPICAL CONCRETE MIXES

Proportions of Mix by Weight	Flow	Air Content		Coefficient of Variation
		Average of Ten Determinations Percent	Standard Deviation Percent	
1:1.53:4.14	55	0.99	0.06	6.5
1:2.20:3.47	55	1.57	0.10	6.2
1:2.80:2.87	55	2.72	0.22	8.1
1:2.20:3.47	85	.89	0.04	4.5
1:2.80:2.87	85	1.31	0.06	4.7
1:2.80:2.87	110	1.15	0.06	4.9

The reproducibility was better for the wetter mixes. This is to be expected since these mixes are more "placeable" and hence specimens of concrete can be rodded into the container more consistently from time to time than can the drier, harsher mixes.

It is evident that increase in the sand-gravel ratio increases the air content of the mix; increase in water content decreases the air content.

4. Comparison of measured with computed values

A comparison was made, in a number of the tests reported in Table 2, between the test results and the corresponding values computed from the weight and apparent specific gravity of the concrete and its constituents. In general, as shown in this table, an agreement exists between the values of air content determined by these two methods.

The relation used in the computation of the air contents given in Table 2 is as follows:

$$V_a = V - \frac{W}{(1 + P_s + P_g + P_w)} \left(\frac{1}{S_c} + \frac{P_s}{S_s} + \frac{P_g}{S_g} + \frac{P_w}{S_w}\right)$$

where

V_a = volume of air in sample in milliliters
V = total bulk volume of sample in milliliters
W = weight of sample in grams
P_s = ratio of sand to cement by weight
P_g = ratio of gravel to cement by weight
P_w = ratio of water to cement by weight
S_c = apparent density of cement, grams/ml
S_s = apparent density of sand, grams/ml
S_g = apparent density of gravel, grams/ml
S_w = density of water, grams/ml

It was found that in the method of computation several steps may be taken to increase the accuracy of the result. The specific gravity of the cement was determined by immersion in water rather than in kerosene, and a value slightly higher than the value obtained with kerosene was thus obtained. In the computation it is also necessary to use the actual proportions of material existing in the sample of concrete rather than the nominal proportions originally weighed for the batch. The difference between the actual and nominal proportions is particularly great when small mixing batches are used, with a resultant relatively large loss of sand and particularly of cement, and water. Where the actual mix present is in doubt an analysis of the concrete, as by the Dunagan method* may be made.

It is seen from the results given in Table 2 that even with these corrections the agreement between the two methods is still not exact, but is reasonably close. The average discrepancy in the two sets of values shown is 5.1 per cent, with a maximum discrepancy of 21 per cent.

SUMMARY

It has been shown that the test method described gives values for air content that agree well with values computed from the carefully measured

*W. M. Dunagan, Proc. Am. Soc. Testing Materials, 31, pt. 1, 383 (1931)

apparent specific gravities of the constituents and the proportions as measured by the Dunagan method.

The method, by reason of the utilization of vacuum, is found particularly suitable for measuring the air contents of aerated concretes where special means must be used to break down the foam developed by the aerating agents.

The grading of the dry materials influences the air content which is evidently influenced more by the relatively large particles in the gravel and sand than by the relatively small particles in the sand and cement. Other factors which may affect the air content, such as the type of mixer and time of mixing, were not considered.

In the limited number of tests made, the air content of concrete without admixtures or additions varied from 0.8 to 2.7 per cent over a fairly wide range in water content and mix proportions. It was found that the intergrinding of the cement and 0.05 per cent of various aerating agents increased the air content of concrete from a value of one per cent to values ranging from 1.5 to 14.0 per cent.

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A Study of the Cause of Nonuniformity in the Compressive Strength of Concrete Pavement Cores*

By V. L. GLOVER†
Member American Concrete Institute

SYNOPSIS

It is common knowledge that the individual compressive strengths of cores from concrete pavements are less uniform than those of cylindrical test specimens made in the laboratory. This more pronounced nonuniformity is believed to be due to variables encountered in field practices which are not present in the laboratory. The Illinois Division of Highways recently had occasion to test a large number of pavement cores and the nonuniformity found in the test results was such as to make these results wholly unsatisfactory for the desired comparisons. A study was made to determine the causes.

An attempt was made to duplicate the nonuniformity known to exist in field concrete, proceeding on the theory that segregation might influence nonuniformity. Results indicate that the position of the coarse aggregate particles in the specimen does not affect greatly the uniformity of the compressive strength; that the amount of coarse aggregate particles in the specimen, that is, the degree to which separation of the aggregate particles and the mortar has taken place, does have an effect; that segregation of the coarser particles of aggregate from the finer ones also has an effect; and that part of the nonuniformity of compressive strength is due to the difference in age of the specimens at the time of test, pavement cores usually being tested at ages considerably greater than the usual standardized age of laboratory specimens.

INTRODUCTION

For a long period of years it has been the practice of the Illinois Division of Highways to take cores from newly constructed concrete pavements to check the depth of the slab. This is in conformity with a specification

*Received by the Institute Oct. 23, 1940.
†Engineer of Materials, Illinois Division of Highways.

provision that the contractor shall be penalized if the depth of the slab is less than a specified minimum. For many years these cores were tested for compressive strength which has afforded the opportunity of studying many thousands of these tests. It has been observed that the compressive strength test on cores gives much less uniform data than any other compressive strength test of concrete made. It is not uncommon to find as much as one hundred per cent difference in the strength of two cores taken from the same pavement, although no visual difference between the two is apparent before testing.

Recently a thorough mathematical analysis was made of the strength tests on approximately 1200 cores taken for a special purpose, and the nonuniformity that exists is amply illustrated by the results obtained. These cores were divided into 81 samples, each containing about 15 specimens. The average range in strength from low to high between specimens in these samples was 3449 psi. Five samples showed ranges from low to high greater than 5000 psi. and only one sample showed a range from low to high less than 2000 psi. Considering collectively the 1186 individual cores represented by the 81 samples, 4 cores showed strengths of less than 3000 psi. even when honeycombed cores were disregarded, and 9 showed strengths greater than 9000 psi. The average strength was 5884 psi. The age of the cores ranged roughly from one to seven years.

There is considerable foundation for the belief that lack of uniformity in core strengths is in some manner connected with the coarse aggregate. It is known from laboratory investigations that failure to control the gradation of the coarse aggregate in individual batches of concrete introduces an element of nonuniformity in the strength results, sometimes sufficient to obscure the particular properties under investigation. It may be significant also that the most uniform strengths in the 81 group samples mentioned above were obtained on cores from an experimental section on which the coarse aggregate was furnished in two sizes.

It was thought that the compressive strength of cores might be influenced by segregation of the coarse aggregate in one or more of the following ways: (1) The coarse aggregate particles might occupy different positions in the cores, causing these cores to react differently in the compressive test. (2) The coarse aggregate particles might be segregated from the mortar, causing some of the cores to be predominantly mortar and others to contain more than the normal amount of coarse aggregate. (3) The larger sizes of coarse aggregate might become separated from the smaller sizes, causing nonuniform gradation.

The condition described under (1) was investigated first. The data obtained, however, did not explain the problem; they did, however, indicate the desirability of investigating the conditions described under (2)

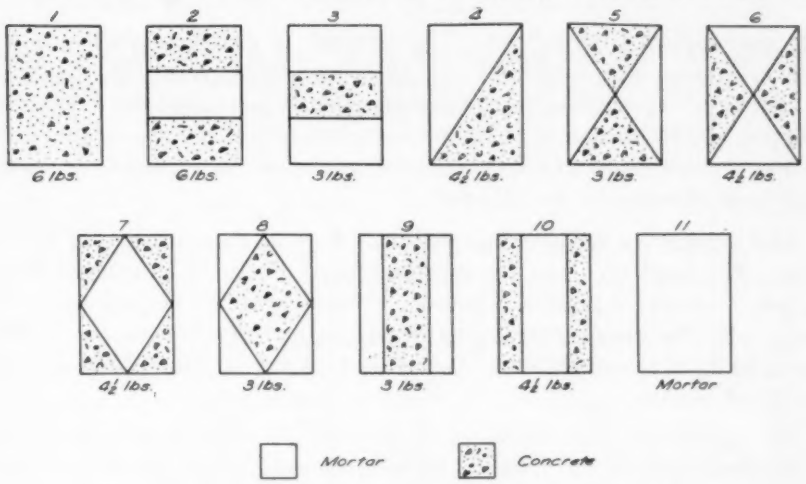

Fig. 1—Specimen type numbers and amount and distribution of coarse aggregate in each.

and (3). Due to the fact that the two parts of the investigation were conducted separately, it was not possible to use materials from the same sources in all of the tests or to use the same test ages. For this reason, the two parts will be discussed separately.

EFFECT OF POSITION OF COARSE AGGREGATE PARTICLES ON COMPRESSIVE STRENGTH

Test specimens, materials, and mixtures

In order that the laboratory tests might be as nearly comparable as possible with tests of pavement cores, 5 x 8 in. cylindrical specimens were used. The various positions of the coarse aggregate particles in the specimens were obtained by using a mortar mixture and placing the coarse aggregate particles by hand at the desired locations while the molds were being filled, making certain that at all times each coarse aggregate particle was surrounded with mortar.

To facilitate the placing of the coarse aggregate particles by hand and to make it possible to duplicate the various specimens, it was found desirable to use particles ranging from 1 to $\frac{3}{4}$ in. In this way specimens were provided which not only represented extreme conditions of segregation of coarse aggregate from the mortar, but also the extreme condition of segregation of a single size of particle from the remaining sizes.

Fig. 1 shows cross sections of the various types of specimens used, one specimen of each being made at the same time and termed a set. One specimen with the coarse aggregate distributed uniformly in the mortar (Type 1) and one specimen consisting entirely of mortar (Type

11) were included in each set. The amounts of coarse aggregate used in the specimens are shown in the sketches; these were the maximum amounts which could be placed conveniently at the desired locations (Types 2 to 10, inclusive). Twelve complete sets of the specimens shown in Fig. 1 were made. Gravel was used as the coarse aggregate in six sets and crushed stone in the other six.

The aggregates used in the specimens were all taken from the laboratory stock kept on hand for investigational work. The cement was a brand of standard portland cement. The mortar in the specimens was essentially the same as that used in pavement concrete, the proportions being 94 lb. of cement, 220 lb. of sand, and 5.4 gal. of water, yielding 2.595 cu. ft. of mortar.

The specimens were cured for 63 days at a temperature of 70 F. and a relative humidity of from 95 to 100 per cent, after which they were capped and tested for compressive strength. They were tested at an age of approximately two months rather than at an earlier age because cores received from the field are seldom less than two months old when tested.

Results of the tests

The compressive strengths of the individual specimens and the average strengths by types and sets of specimens are shown in Table 1. Disregarding the strength of the mortar specimens, it is seen that the individual strengths were very uniform. If the average mortar strengths are disregarded, the average strengths by types are about as uniform as the average strengths by sets. Nothing unusual was found in the manner of failure of the specimens, 110 of them showing normal conical breaks and 22 showing diagonal breaks. Therefore, it appears that nonuniformity of compressive strength, such as usually found in the testing of pavement cores, is not caused by variation in the position of the coarse aggregate particles.

The mortar strengths are considerably greater than the concrete strengths, and the strengths of the concrete are lower than are usually obtained from concrete made with a mortar of standard proportions (Table 1). It was fully expected that the mortar specimens would exceed the concrete specimens in strength. It has long been known that the more or less accepted principle that "concrete is of essentially the same strength as its mortar" is true only when the concrete cylinders are tested in comparison with 2 x 4-in. mortar cylinders. However, the fact that mortar shows a higher strength than concrete when equal size specimens are used indicates that the amount of coarse aggregate rather than its position in the specimens might be one of the causes of nonuniformity. Also, the fact that the concrete specimens did not develop the strength

TABLE 1.—COMPRESSIVE STRENGTHS IN POUNDS PER SQUARE INCH OF INDIVIDUAL SPECIMENS AND AVERAGES BY SETS AND BY TYPES OF SPECIMENS. AGE OF SPECIMENS—63 DAYS

Specimen Type Number	Set Number						
	1	2	3	4	5	6	Av.
GRAVEL COARSE AGGREGATE							
1	4269	4004	3984	3884	4183	5079	4234
2	2937	4263	4248	3897	4487	4619	4075
3	4247	4786	5030	4722	4379	4188	4559
4	4254	4429	3832	4292	4443	4222	4245
5	4467	4360	4584	3852	4250	5221	4456
6	4639	4047	4357	4253	3924	4734	4326
7	4340	3769	4283	3932	3683	4621	4105
8	4612	4547	4199	4105	4578	4863	4484
9	4183	4357	4005	4091	4043	4787	4244
10	3587	4523	3699	3300	5309	5207	4271
11	5824	5069	5446	4483	5128	6174	5354
Average	4305	4378	4333	4074	4401	4883	4396
CRUSHED STONE COARSE AGGREGATE							
1	3807	3708	3448	3810	3887	4217	3813
2	4570	3135	4080	3421	4569	3350	3854
3	3808	4234	3797	3930	4116	3523	3901
4	3449	4244	3389	3280	4381	4057	3800
5	3678	4284	4531	3707	3495	3810	3918
6	3699	3753	3740	3439	3571	3615	3636
7	4058	4214	4092	3607	3969	3225	3861
8	4072	4248	3223	4067	3781	3884	3879
9	3798	3524	3718	3955	3538	3489	3670
10	3749	3902	3815	3223	3777	3617	3680
11	5452	5464	5197	4799	5112	4780	5134
Average	4013	4065	3912	3749	4018	3778	3922

anticipated might have been caused by the use of only one size of aggregate in the specimens, indicating that variations in gradation also might be a cause of nonuniformity.

In view of the results obtained, it was decided to eliminate from further consideration the position of the coarse aggregate particles in the specimens and to make additional tests to study the effect of variable amounts and gradations of coarse aggregate.

TABLE 2.—MATERIALS AND MIXTURES

GRADATION OF COARSE AGGREGATE

Item	Standard				Gap			
Passing Sieve	1½"	1"	½"	No. 4	1½"	1"	½"	No. 4
Percent.....	100	75	20	0	100	44.4	44.4	0

NOTE.—Variation from standard gradation shown below.

STANDARD QUANTITIES PER BAG OF CEMENT

Sand—220 lb. (Specific Gravity, 2.65)
Water—5.4 gal.
Yield—2.595 cu. ft. of mortar

STANDARD AMOUNT OF COARSE AGGREGATE PER BAG OF CEMENT ADDED TO THE MORTAR

Gravel—368.6 lb. (Specific Gravity, 2.70)
Yield—4.78 cu. ft. of concrete
Stone—335.5 lb. (Specific Gravity, 2.67)
Yield—4.61 cu. ft. of concrete

NOTE.—Variation from standard amount shown below.

AMOUNT AND GRADATION OF COARSE AGGREGATE IN THE SPECIMENS

Type Number	Gradation of Coarse Aggregate	Amount of Coarse Aggregate
11	Mortar	None
12	Standard	Standard
13	Standard	¼ of Standard
14	Standard	½ of Standard
15	Standard	1¼ of Standard
16	Standard	1½ of Standard
17	No. 4 to ½"	Standard
18	No. 4 to 1"	Standard
19	½" to 1½"	Standard
20	1" to 1½"	Standard
21	Gap	Standard

EFFECT OF AMOUNT AND GRADATION OF COARSE AGGREGATE ON COMPRESSIVE STRENGTH

Specimens, materials, and mixtures

Cylindrical specimens 5 x 8 in. were used. The cement, sand, and gravel were from the same laboratory stock as in the previous tests; the crushed stone was from another source. The proportions of the mortar were the same as in the previous tests, but the coarse aggregate was varied both as to amount and gradation (Table 2.).

The concrete was mixed by hand, the size of the batch being sufficient to make two cylinders. Two cylinders of each type were made each day specimens were prepared. To make possible exact duplications of the

gradations used, the coarse aggregate was separated into various sizes and before adding it to the mortar the correct amount of each size was weighed out separately and the several sizes recombined before mixing.

Eight specimens of each type of mixture were made. They were cured in the moist room at 70 F. and a relative humidity of from 95 to 100 per cent until removed and capped for testing. Two specimens of each type were tested at the age of 14 days to provide an early comparison of the mixtures and it was intended to test the remaining six specimens of each type at the age of 63 days as in the previous tests. It was impossible, however, to follow this program due to the installation of a new 400,000-pound hydraulic testing machine underway at this time, and the specimens were therefore tested at the age of about 120 days.

Results of tests

The results of the 14-day and 120-day tests are shown in Tables 3 and 4, respectively. It is noted even from a superficial examination that considerable nonuniformity of compressive strength existed between the different types of mixtures, even though there was in general good uniformity in strength between specimens of the same type in the various sets. It is also seen that there is good agreement between the 14-day and 120-day strengths, although the latter, as far as the average strengths are concerned, would be expected to show most nearly the true relation between the various mixtures because of the greater number of specimens tested.

Referring to the data in Table 2, it is seen that Types 11, 13, 14, 12, 15, and 16 mixtures, in the order named, represent a series in which the amount of coarse aggregate was gradually increased from zero to one and one-half times the standard amount. Likewise, Types 17, 18, and 12 mixtures represent a series in which the maximum size of the particles of coarse aggregate was gradually increased. In both series, the coarse aggregate was well graded between the minimum and maximum sizes. Finally, the Types 19, 20, and 21 mixtures represent a series in which the coarse aggregate was of irregular gradation. In the last two series mentioned the standard amount of coarse aggregate was used. The average compressive strengths from Tables 3 and 4 are shown in Table 5 in accordance with this grouping.

In the first series of mixtures (Types 11, 12, 13, 14, 15, and 16) Type 11, the mortar mixture, produced the highest strength and the addition of coarse aggregate caused a reduction in strength roughly proportional to the amount added, up to the point where the mixture contained the standard amount. The addition of coarse aggregate beyond this resulted in a very sharp decline in strength, no longer proportional to the amount

TABLE 3.—COMPRESSIVE STRENGTH IN POUNDS PER SQUARE INCH AT THE AGE OF 14 DAYS

Type Number	Gravel Concrete			Crushed Stone Concrete		
	A	B	Av.	A	B	Av.
11	4450	4398	4424	5047	5183	5116
12	3385	3416	3401	4508	4709	4609
13	4243	3467	3855	4636	4220	4428
14	3522	3489	3506	4655	4287	4471
15	3070	3079	3075	2823	3755	3289
16	3002	2676	2839	3072	3230	3151
17	3116	3736	3426	3900	3748	3824
18	2476	2664	2570	3577	3515	3546
19	1904	2604	2254	3534	2686	3110
20	2321	2537	2429	3130	2771	2951
21	2977	2820	2899	3857	3291	3574

TABLE 4.—COMPRESSIVE STRENGTH IN POUNDS PER SQUARE INCH AT THE AGE OF 120 DAYS

Type Number	Set						
	C	D	E	F	G	H	Av.
GRAVEL CONCRETE							
11	6470	6585	6770	6665	6985	7500	6829
12	4950	4965	5450	4975	7085	4325	5291
13	5905	6205	6150	6320	6430	6860	6312
14	5760	5300	5490	6120	6280	6388	5890
15	4310	5020	3745	3915	3320	3940	4042
16	3650	3930	4150	4055	3360	4125	3878
17	5225	5515	4660	5455	5238	5313	5234
18	5495	4985	5185	3995	5495	5240	5066
19	5100	5195	3860	4425	4330	4700	4601
20	3820	4295	3815	4250	4058	4405	4107
21	6000	5565	4995	4785	5070	4565	5163
CRUSHED STONE CONCRETE							
11	6070	6395	7360	6825	7615	7600	6977
12	6480	6620	5570	5800	5300	6755	6087
13	6470	6190	6955	6815	6470	7585	6747
14	5770	5865	7200	6430	6170	6780	6369
15	4435	4815	4557	4670	3870	4605	4492
16	4835	3815	4265	3610	4155	4505	4197
17	4755	5150	6055	6800	5835	7275	5978
18	5615	5780	6400	5935	7365	6470	6260
19	5335	4805	4955	4550	4945	5165	4959
20	5175	5050	4300	3835	5357	4427	4690
21	6385	6385	5460	5830	6855	6115	6172

TABLE 5.—RELATION OF AMOUNT AND GRADATION OF THE COARSE AGGREGATE TO COMPRESSIVE STRENGTH OF THE CONCRETE

Explanatory Notes and Numerical Quantities		Type No.	Average Compressive Strength psi.					
			14-Day Age			120-Day Age		
			Grvl.	Stone	Av.	Grvl.	Stone	Av.
Variable Amount of Coarse Aggregate—Standard Gradation								
Relative Amounts of Coarse Aggregate	0	11	4424	5116	4770	6829	6977	6903
	¼	13	3855	4428	4142	6312	6747	6530
	½	14	3506	4471	3989	5890	6369	6130
	*1	12	3401	4609	4005	5291	6087	5689
	1¼	15	3075	3289	3182	4042	4492	4267
	1½	16	2839	3151	2995	3878	4197	4038
Standard Amount of Coarse Aggregate—Uniform Gradation								
Maximum Size Coarse Aggregate	½"	17	3426	3824	3625	5234	5978	5606
	1"	18	2570	3546	3058	5066	6260	5663
	1½"	12	3401	4609	4005	5291	6087	5689
Standard Amount of Coarse Aggregate—Irregular Gradation								
Sizes of Coarse Aggregate See Table 2	½" to 1½"	19	2254	3110	2682	4601	4959	4780
	1" to 1½"	20	2429	2951	2690	4107	4690	4399
	Gap	21	2899	3574	3237	5163	6172	5668

*The numeral "1" signifies the standard amount of aggregate.

added. While some of the 14-day strengths are slightly inconsistent, there is no doubt that variable amounts of coarse aggregate produce variable compressive strengths in concrete, even though the mortar remains constant in quality.

In the second series of mixtures (Types 12, 17, and 18), the size of the largest particles did not matter greatly as long as the amount of the coarse aggregate remained the same and graded coarse aggregate was used, although again the 14-day strengths are slightly inconsistent.

In the third series (Types 19, 20, and 21) only Type 21, the gap gradation, produced results in reasonable conformity with those obtained with the standard amount and gradation of coarse aggregate, showing that the smaller sizes of coarse aggregate are of great importance.

Considering the three series collectively, only the Types 12, 17, 18, and 21 mixtures may be said to have produced standard strength. The standard amount of coarse aggregate was used in all of these, reasonably

graded to the maximum size except for the gap gradation used in the Type 21 mixture. The Types 11, 13, and 14 mixtures produced strengths higher than the standard, but these are uneconomical because of their low coarse aggregate content. Types 15, 16, 19, and 20 mixtures produced inferior strengths, either because of too much coarse aggregate or a lack of the smaller sizes.

It appears therefore that any segregation of coarse aggregate from the mortar and any segregation of the larger coarse aggregate particles from the smaller ones during the process of constructing a pavement might be responsible for some of the nonuniformity found in pavement cores. Cores, being very small samples from a pavement, would undoubtedly be, at times, predominantly mortar, in which case high strengths would be obtained; or they might contain too much coarse aggregate or be deficient in the smaller sizes, causing low strengths. In fact, a core might contain any combination of coarse aggregate and mortar, especially in the older pavements in which the concrete mixtures were given less attention than they are now.

Considering the individual strengths in Table 4, some of them are nearly twice as high as others, even disregarding the mortar specimens (compare for example Types 12 and 16). In fact the data in this table have very much the characteristics of core test data and it is believed that they go a long way toward explaining the nonuniformity usually found in such data. However, the same could scarcely be said of the 14-day strengths shown in Table 3 and the conclusions drawn above probably could not have been arrived at on the basis of tests at early ages. As a matter of fact core tests in one instance at least were made at the age of 14 days and the uniformity of strength was almost comparable with that of controlled laboratory specimens. It appears, therefore, that age is a factor in causing nonuniformity.

EFFECT OF AGE ON UNIFORMITY OF COMPRESSIVE STRENGTH

Numerical measure of uniformity of strength results

In much of the investigational work performed in the past, scant consideration has been given to any other function of a group of test results than the mean or average value. The significance of the average value needs no explanation. It is the best known value of the particular property measured through the tests.

It is readily seen, however, that in cases where the individual test results show considerable dispersion, the average value may fall short of furnishing the desired information. In such cases, the range from the highest to the lowest of the individual test results has sometimes been used as an indication of the uncertainty which exists if the average value is considered applicable to all individuals represented by the specimens

tested. Sometimes, in an effort to obtain what is considered as the best average value from a group of test data, the most extreme values of individual test results have been arbitrarily discarded, which practice cannot be justified unless it is known in the first place that the specimens giving the extreme results were not representative samples. Reasonably definite information as to how much lower or higher than the average the test results for any individual specimen might be, if it were possible to make a very large number of tests, would be valuable additional information in all cases and almost indispensable in some. Such information is supplied by the standard deviation of test results.

Standard deviation of compressive strength results at various ages

It is not within the scope of this report to give a complete discussion of the standard deviation or to explain the methods whereby the standard deviation may be estimated. Reference may be made to any good book on the subject.(*) For the purpose of this discussion it is sufficient to know that the standard deviation is the root-mean-square deviation of the test results from the average value; that in normally distributed statistical data of any kind, practically none of the individual values will deviate from the average by more than three times the standard deviation; and that from past experience in testing, normal distribution of test results is known to exist in the compressive test data for concrete. The following example will illustrate the use of the standard deviation as a numerical measure of uniformity of test results:

Suppose two groups of test results representing similar products, give the same average value such as, for example, 5000 psi. If nothing else is known, it would be natural to conclude that the two products represented by the test data were of equal quality. If, however, it is also known that the two products have standard deviations of 300 and 800 psi., respectively, it is immediately known that practically all individuals in one product would show strengths falling between $5000 \pm 3 \times 300$, or between 4100 and 5900 psi.; and that practically all the individuals in the other product would show strengths falling between $5000 \pm 3 \times 800$, or between 2600 and 7400 psi. It is obvious that the two standard deviations measure the uniformity of the respective products and are valuable information in addition to the average.

Several available groups of compressive strength data were examined to obtain estimates of the standard deviation at various test ages. The specimens represented by these groups were all made from mortar designed for standard strength and no known deviation from standard methods existed in their making, curing, and testing.

(*) For example, "The Mathematical Theory of Probabilities and Its Application to Frequency Curve and Statistical Methods," by Arne Fisher; and "Economic Control of Quality of Manufactured Product," by W. A. Shewhart.

TABLE 6.—RELATION OF STANDARD DEVIATION TO AGE OF THE CONCRETE OBTAINED FROM COMPRESSIVE STRENGTH DATA

Kind of Specimen	Age in Days	Number in Group	Av. Comp. Strength psi.	Standard Deviation	
				psi.	Percent
6″x 12″ Concrete Cylinder	7	30	4315	290	6.7
6″x 12″ Concrete Cylinder	14	30	5422	244	4.5
6″x 12″ Concrete Cylinder	28	30	5925	316	5.3
6″x 12″ Concrete Cylinder	43	60	4915	435	8.9
5″x 8″ Concrete Cylinder	63	132	4159	404	9.7
5″x 8″ Concrete Cylinder	122	132	5425	533	9.8
2″x 4″ Mortar Cylinder	153	84	7452	547	7.3
6″x 12″ Concrete Cylinder	260	30	7580	651	8.6
2″x 4″ Mortar Cylinder	286	120	6900	579	8.4
Pavement Cores	See Note	1186	5884	1013	17.2

NOTE.—All of the specimens were made from the same general class of concrete obtained by using a standard strength mortar in the mixtures and were (except the cores) subjected to the same causes of variability in making, curing, and testing. The pavement cores undoubtedly were subjected to more severe conditions of variability than the laboratory controlled specimens and these, in addition to increase in age, are no doubt reflected in the high value of the standard deviation. The pavement cores ranged in age approximately from 1 to 7 years. The standard deviations are the corrected average values obtained from subgroups of six specimens, except in the case of the pavement cores which subgroups of 15 were considered. For methods employed, see W. A. Shewhart, "Economic Control of Quality of Manufactured Product."

Known differences however did exist between the groups which were sufficient to produce some inconsistencies in the age-strength relation, and differences existed within individual groups sufficient to cause some variation in average strength between subgroups of specimens. The major differences between groups were the test age, the size of the test cylinder, the brand of cement, the source of aggregates, and the type, amount, and gradation of coarse aggregate. Known variables within respective individual groups were the type, amount, or gradation of coarse aggregate, the amount of admixture used, the method of surface treatment, and moisture content of specimens at the time of testing.

In examining the various groups of data, it was found that with the exception of the test age the variables between groups and within individual groups, although sufficient to affect the average strengths, apparently did not cause variation in the values of the standard deviation beyond that which normally would be expected in strictly comparable groups of compressive strength data. In fact no consistent variation was found in the values of the standard deviation other than an increase with age of the concrete.

The results of this study are shown in Table 6, in which is included also the estimate of the standard deviation of a large group of pavement core strengths. The increase of the standard deviation with age of the concrete is reasonably well defined as far as the absolute values are

concerned. Since it is often desirable to express standard deviations in relative terms, these are also shown in percentages of average strength. These percentages are influenced somewhat by the inconsistencies in the age-strength relation between the groups and would not be expected to show a relation with age as consistent as that shown by the absolute values. Nevertheless, these percentages confirm definitely that the standard deviations are lower at early ages when most laboratory specimens are tested than at later ages when most pavement cores are tested, and that nonuniformity in compressive strength increases with the age of the concrete, whether viewed from the standpoint of absolute or relative values of the standard deviation.

The specimens considered herein, regardless of existing differences, may nevertheless be considered as having been made from the same general class of mixtures. For this particular class of mixtures, the values in Table 6 further indicate that the standard deviation is not influenced greatly by the particular make-up of the concrete and that in addition to being a function of the age of the concrete it may be almost entirely a function of the differences between individual specimens in any given group introduced during the entire process of making, curing, and testing of the specimens.

As more data become available, it will be possible to plot a rather accurate curve showing the relation between age and standard deviation for compressive strength of concrete produced under laboratory control and of the general quality class obtained by using standard strength mortar in the mixtures. A curve of this nature would have many uses. It could be used for determination of the decrease in uniformity in strength occasioned by deviation from laboratory control. It would also make possible the development of methods for judgment of individual strength values and average strength values at the time of making the tests; some work along this line is in progress at the present time.

CONCLUSIONS

It is believed that the data and discussion presented herein warrant the following conclusions:

(1) The position of the coarse aggregate particles in the test specimen has a negligible effect on uniformity of strength.

(2) The nonuniformity of compressive strengths usually found in core test data as compared with that obtained in the laboratory is due (a) to segregation of the coarse aggregate particles from the mortar (variable amount of coarse aggregate in the specimens), (b) segregation of the larger coarse aggregate particles from the smaller ones (irregular gradation of the coarse aggregate particles in the specimens), and (c) testing

of cores at greater ages than usually employed in the testing of laboratory specimens.

(3) The furnishing of coarse aggregate in separated sizes and recombining these sizes during proportioning of the materials would go far toward elimination of the nonuniformity due to segregation in the particles of coarse aggregate.

Discussion of this paper should reach the ACI Secretary in triplicate by Mar. 1, 1942, for publication in the JOURNAL for June, 1942

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Discussion of a paper by V. L. Glover:

A Study of the Cause of Nonuniformity in the Compressive Strength of Concrete Pavement Cores*

DISCUSSION BY VALENS JONES, S. P. WING and V. L. GLOVER

By VALENS JONES†

Mr. Glover is to be commended on his interesting paper concerning nonuniformity of core strengths and his attempts to find by laboratory tests the cause of variations. His statement that cores are more variable than laboratory cylinders is substantiated by the coefficient of variation shown for cores in Table 2 of the 3rd edition of the Concrete Manual of the Bureau of Reclamation. It is agreed that a coefficient of variation of 17 percent is high, and that there should be a plausible explanation of the cause. It is not agreed, however, that variation in the percentage of coarse aggregate is entirely or even primarily responsible for the difference in uniformity of the cores as compared with concrete cylinders made in the laboratory.

A laboratory specimen is fabricated under more rigidly controlled conditions than field concrete. Close control is exercised not only of mix proportions, including amount and grading of coarse aggregate, but also of mixing, placing, and curing. There are many sources to which strength variations may be traced, and variation in the amount and grading of coarse aggregate is but one of these sources. It is evident from the author's text and third conclusion that many of the cores represented pavement made with pit run coarse aggregate. Under these circumstances, variable grading of the pit and segregation of fine sizes would cause nonuniformity of concrete at the mixer. When concrete is nonuniform at the mixer, what happens during subsequent handling? Many of the operations between the mixer and the forms can cause nonuniformity of the finished product, and none of these

*ACI JOURNAL, Nov. 1941; *Proceedings* V. 38, p. 133.
†Denver, Colo.

operations can be relied on to transform a widely fluctuating mixer product into smooth, uniform concrete in its final position in the forms. In fact, all handling operations must be carefully managed if segregation and nonuniformity are to be avoided.

Unfortunately the author has given so few data relative to the concrete cores that it is impossible to judge what portion of their variation in strength may be attributed to the coarse aggregate and what portion might be due to variation of items that were not mentioned, such as: water-cement ratio, cement content, and workability of the fresh concrete; cement from a different bin or carload; kind and amount of curing; the season of the year when concrete was placed; and special conditions, such as alkali exposure. Also, no data were given to indicate that such an extreme variation of the amount of coarse aggregate as one-fourth to one and one-half times the standard amount was found in the cores that were tested, and it is doubted that this variation could occur without causing such difficulty that correction would shortly be made. Inspection by competent men on the job would not allow segregation to this extent in an appreciable area of pavement; hence, cores taken at random would not be apt to show as much variation in strength as that shown in Table 5 because of variation in the amount of coarse aggregate. Some of the laboratory mixes reported are considerably outside of the range of mixes that would be practical on the job, and there is doubt that they constitute a sufficient basis for Mr. Glover's conclusion that the variation in core strengths is due to the variable amount of coarse aggregate in the specimens.

It is not unusual to find 14 out of 1,200 test values outside of the range of 3,000 to 9,000 psi when the standard deviation is 1,013 psi, and no indication is given that the data were obtained under controlled conditions. This is only a little over one per cent; whereas, for data obtained under controlled conditions, statisticians indicate that about one-fourth of one per cent of the values are expected to lie outside of the range obtained from the average plus and minus three times the standard deviation, which would be 3 outside of a range from 2,840 to 8,920. It is to the author's credit, however, that he has presented his data in a standard statistical manner so that the reliability of the average strength can be readily ascertained.

Throughout his paper strengths are shown to four significant figures for individual and average values. It is not always realized that nothing is gained by this refinement in reporting compressive strength. The average standard deviation for groups of six specimens shown in Table 1 is 404 psi (from Table 6), and there is only a 50-50 chance that future averages obtained under the same conditions will be within 135 psi of

making or curing of these cylinders produce significant changes in the averages shown.* In other words, when there is doubt about the last 135 pounds, a difference of a single pound has no significance. Another approach is that when a measure of dispersion such as standard deviation is given to the recommended three places, it is understood that about two-thirds of the test values were between the limits obtained by adding the standard deviation to and subtracting it from the average. Keeping in mind that such a dispersion exists, your judgement of concrete quality is not altered by reducing an average strength from 5,764 to 5,760 psi. The amount of information lost by such rounding of figures is insignificant; and, with fewer significant digits, mistakes are less likely and time is saved.

The use of pit run coarse aggregate has a long record of contributing to nonuniformity of the mix. Consequently, for all except the smallest jobs it is preferable to separate the coarse aggregates into convenient size fractions and recombine them in accurate proportions at the time of mixing. However, aggregates that are not carefully handled may segregate to such a degree that uniformity will be greatly impaired.† It appears that uniformity of the mix may best be assured by arranging, where practicable, to do the final screening over bins at the batching plant.

By S. P. WING‡

The use, by the author, of statistical measures to define the nonuniformity of pavement cores is another welcome indication that methods for the control of tests similar to those given in the ASTM *Manual on Presentation of Data* are increasingly being adopted. It is believed that as engineers become more familiar with the advantages, not forgetting the limitations, of applying statistical analysis to concrete technology, that order will be brought into much conflicting published data concerning the properties of concrete and that there will be a better realization of the variables which have to be controlled and the number of tests which are needed if reliable conclusions are to be obtained.

In a personal letter to the writer, the author gives the reasons for the grouping of the pavement cores into 81 samples of about 15 cores each. Twenty-seven different highway sections, built under approximately the same specifications, were each sampled for strength in three different years with 15 drilled cores. Although for any given section, the means of these tests provided but three points on the strength-age relationship, as for example, for 1, 2, 3 or 3, 4, 5 years, the series as a whole had

*Computed by the method shown in Supplement A (1935) of the 1933 A.S.T.M. Manual on Presentation of Data. The value obtained for limits of uncertainty of an average computed by this method is a little greater (about 10 percent in this case) than the probable error of the average, a measure of reliability that has lost favor in recent years.

†The effect of segregation in coarse aggregate and corrective measures are presented by Lewis H. Tuthill in a discussion of "Concrete Control" by I. L. Tyler, *Transactions* of A.S.C.E., v. 106 (1941), p. 1272.

‡Engineer, Bureau of Reclamation, Denver, Colo.

data up to an age of seven years. Notwithstanding this large amount of material, no consistent long term increase in strength of concrete with age was found, the points for various ages for any given section varying from the mean for that section within the limits of expectancy as defined by the standard deviation of the various sections. From the analysis of the 1186 cores, with an average strength of 5,885 lb., the estimated standard deviation was found to be 1,013 lb. or 17.2 per cent. This index of variation compares favorably with an earlier series of 2100 highway core tests aged one to eleven years, reported by A. N. Johnson* which had an average strength of 4,080 lb. and a standard deviation of 910 lb. (22.2 per cent) and for which likewise little variation of strength with age was found. However, if either of these standard deviations be compared with those found in the laboratory in which the standard deviation for specimens made from different batches on the same day is of the order of 5 per cent and for batches made over a considerable period about 10 per cent (Lewis Institute reported 500 samples of 10 specimens from different batches with estimated standard deviation of 10 per cent,)† it is evident that in practical construction new variables productive of variability are introduced which in themselves have a standard deviation of the order of 10 to 15 per cent. The author successively investigates the hypotheses that these variables may be first, nonuniformity in the position of the coarse aggregate, second, variation in the amount of coarse aggregate, and thirdly, the effect of age.

Tests representing the effect of segregation were simulated by placing the aggregate by hand in various patterns in the cylinders, and were given in Table 1. Eleven types were made up each day, and the series was repeated six times. The means for the various types excluding Type 11 of pure mortar, had a standard deviation of about 130 lb. (3.0 per cent) a value without significance considering that the experimental error or the ability to reproduce the results obtained for any one specimen appears to have been about 10 per cent. The author, therefore, answers in the negative his hypothesis that the position of the coarse aggregate caused nonuniformity. However, if it be considered that a similar series of tests of a normal mix made under reasonable laboratory control should show a standard deviation of not more than 5 per cent, it would appear that either the hand placing of the aggregate or its segregation in patterns was responsible for an added factor of variability of about 8 per cent. One other interesting fact, indicated by the data of Table 1 should be mentioned. The standard deviation of the means of the sets of eleven, made on different days is about 7 per cent, a statistically significant value. If day to day variations in the

*Johnson, A. N., Strength Characteristics of Concrete as Indicated by Core Tests. *Public Roads*, 9:148 Sept., 1928.
†Ibid.

strengths for laboratory concrete, it would not be surprising if the varying conditions of exposure and climate to which concrete is subjected in the field should prove to be a major factor in producing variability.

Table 2 shows conclusively that it is possible by varying the amount of coarse aggregate in a mix to produce material changes in strength; for these tests an increase of one per cent in the amount of coarse aggregate decreases the strength by 0.45 per cent. This may be compared to increasing the water-cement ratio by one per cent, which for a mix of this character would decrease the strength by 1.2 per cent. But before the author can ascribe to changes in the amount of coarse aggregate an important part in the increased variability found in field concrete, it is necessary to show that large variations actually occurred in the amount of coarse aggregate contained in his cored test specimens. No data of this sort were available, but if the standard deviation of the amount of coarse aggregate contained in a specimen be estimated at 10 per cent, the corresponding strength change is about 5 per cent. This is but a small part of the variability for which explanation is sought.

Lastly, the author discusses the effect of age on uniformity. It is not clear whether he has in mind that strong concretes may have higher standard deviations measured in pounds than weak concrete, the same concrete being involved, and the only difference being that of the length of curing period, or whether he refers to what the writer would call the hazards of concrete life; that is, the older the concrete, the more likely for it to have been exposed to various types of weather and sub soil curing which increase the variability of test results, regardless of its average strength. There is considerable experimental and theoretical grounds for believing that the standard deviation expressed as a per cent is a constant independent of strength but having a value characteristic of a given laboratory or test procedure. Within limits such factors as a given effective cylinder eccentricity, increased tamping, etc., will produce a given percentage strength change regardless of absolute values. Illustrating this, an analysis of three series of tests of 6 x 12-in. cylinders cured 3, 7 and 28 days, each series consisting of three cylinders from each of 28 different batches gave the following estimated standard deviations with respect to the individual batch means:

Days of Cure	Mean Strength Lbs.	Estimated Standard Deviation with respect to Batch Mean	
		Lbs.	Per Cent
3	1216	30	2.5
7	1862	41	2.2
28	4246	85	2.0

Since the differences in these percentage values are not significant, 2.3 per cent can be taken as a standard of variability for the test technique of this particular laboratory for cylinders taken from the same batch, regardless of their absolute strength. On the other hand, it is known that small differences in curing conditions, particularly at early ages, create large changes in strength, and this factor in itself seems adequate to explain much of the variation in the strength of cylinders taken from concrete poured in the open.

Finally, the writer wishes to suggest one factor not mentioned by the author which may well be a major factor in explaining the difference in uniformity found within a series of laboratory tests and that found for cored cylinders; the cement itself. The writer has had access to the results of 28 day acceptance tests made on more than a hundred bins of cement purchased from a number of suppliers under a single specification. Each four hundred barrels of cement entering the bins was sampled, and five samples were blended to provide a strength test. For the entire series the standard deviation of these blends representing 2000 bbl lots was more than 20 per cent and for individual suppliers ranged from 10 to 17 per cent. While it is possible that had the tests been made after one year of constant curing instead of after 28 days the standard deviation measured in percent would have been found considerably smaller, and while it is certain that the handling of the cement from bin to mixer would reduce the job variability of these acceptance test values there seems to be little doubt that where large yardages of cement are involved, the cement itself is an important cause of variation in the finished product. For example, data presented in the Wheeler Project report of the T. V. A.* indicates a standard deviation in the 28 day strength of concrete cylinders made from lots of cement purchased from the same supplier of about 9 per cent based on about 20 lots and 80 observations.

Although the importance of the various factors, which combine to give an overall standard deviation, vary from job to job, it is thought that Table A showing the writer's estimate of the individual standard deviations of the various items which combine to give the 17 per cent variability of pavement cores found by the author presents an approximate picture of their relative importance. It is to be noted that since variability factors combine in accordance with "the square root of the sum of their squares" to give the overall coefficient, the elimination of any single cause of variability produces but a small change in the overall value.

*United States Tennessee Valley Authority. *The Wheeler Project*, p. 179.

TABLE A—ESTIMATE OF MAGNITUDE OF FACTORS CAUSING STRENGTH VARIATIONS IN HIGHWAY TEST CORES

Item Causing Variation	Standard Deviation (per cent)	Mean Square	Sub Totals Mean Square
1. Capping	1	1	
2. Strength of particles of coarse aggregate	1	1	
3. Tamping	1	1	
4. Misc., including mixing	1.5	2.2	
Within Batch	2.3%	5.2	5.2
5. Grading of fine aggregates	2	4	
6. Water-cement ratio	2	4	
7. Location in curing room	3	9	
8. Misc., including temperature of mixing water	2	4	
Batch to Batch	4.6%	21	21
Total Laboratory Items	5%		26.2
9. Quantity of coarse aggregate	5	25	
10. Free water in field aggregate	6	36	
11. Climatic exposure	8	64	
12. Cement from various bins	8	64	
13. Misc.	8	64	
Field Items	16%	253	253
Total Standard Deviation	17%		279.2

The table likewise indicates that the accurate determination of the average strength of concrete placed in the field depends more upon the number and location of the localities sampled than upon details of the laboratory test procedure, and likewise that in designing trial mixes or in testing cement little is gained in testing more than one sample from a single batch, the large factor causing variability in results being the batch to batch variation. This has repeatedly been pointed out by others.*

To conclude this discussion, Table B is a tabulation of standard deviations, expressed as percents for typical classes of concrete work taken from actual structures.

By making use of the tabulated data it is possible to fix an average strength of concrete for field use such that any desired percentage of cores taken from the job will not fall below a given specified strength and thus meet the requirements of Section 327-8A of the Joint Committee Standard Specifications for Concrete and Reinforced Concrete, June, 1940, which determines design unit stresses.†

* Leavitt, H. W. A Study of Plastic Mortar Cubes. *A.S.T.M. Part I, Committee Reports*, 1937, p. 321.

† Bureau of Reclamation. *Concrete Manual*, 3d ed., p. 187 gives additional data and methods of applying statistical control.

TABLE B

Item	Number	Standard Deviation
Drilled cores from highway	4000	17-22% [2]
Drilled cores from highway—3 cores from same cross section in terms of group mean	500 sets	12.5% [1]
Drilled cores from concrete bridge floor slab	300	14% [3]
Typical buildings	1000	20% [4]
Dam 18x36-in. cores	12	15% [5]

[1] Johnson, A. N., op cit.
[2] Glover, V. L. A Study of the Cause of Nonuniformity in the Compressive Strength of Pavement Cores.
[3] Teller, L. W. The Effect of Materials and Methods of Placing on the Strength and Other Properties of Concrete Bridge Floor Slabs. *Public Roads*, 12:237, December, 1931.
[4] Young, R. B. Control of the Quality of Concrete. *American Concrete Institute Journal*, 22:101, 1926.
[5] Tennessee Valley Authority. *The Wheeler Project*, Technical Report No. 2, p. 334.

As far as the writer can see there is little prospect of making a material reduction of these values for jobs which must be carried out over several months, since a variable climate and cement supply must be accepted for most work. On the other hand, from the point of view of laboratory control and research, the estimates of the upper part of Table A indicate the possibility of a material saving in the number of cylinders required to attain a given accuracy providing it is found possible to produce concrete in successive batches, or on successive days as uniform as that which comes from a single batch. This goal does not seem impossible of attainment.

AUTHOR'S CLOSURE—V. L. GLOVER

S. P. Wing and Valens Jones both express doubt as to the existence of the same degree of variability in drilled cores as was introduced in the laboratory specimens represented by Table 4. That segregation of mortar from the coarse aggregate and segregation of the larger coarse aggregate particles from the smaller ones are present to varying degrees in drilled cores may be observed visually during the testing of cores. To advance quantitative proof of the degree to which these conditions are present, however, is by no means an easy matter.

A concrete pavement is not a homogeneous mass of concrete as might be supposed from the appearance of the finished surface. Removal of large sections of slab for the purpose of examining the surface in contact with the subgrade has revealed varying degrees of defects ranging from small indentations to infrequent small spots of severe honeycomb. Severe condition of honeycomb is seldom revealed in drilled cores because it is of infrequent occurrence and because a core is but a very small sample. On the other hand, when cores are drilled at random, some will in the long run be drilled in the honeycombed spots. In addition to the 1186 cores used as an illustration of the nonuniformity which

exists, there were 14 honeycombed cores which were not included because they were not considered sufficiently representative in the comparisons which were to be made. It is seen, therefore, that one out of about 85 cores drilled was visibly honeycombed.

Honeycombed concrete is the result of an extreme condition of segregation of the coarse aggregate particles from the mortar and usually also of an extreme condition of segregation of the larger particles of coarse aggregate from the smaller ones. No condition as extreme as that is represented by any of the data in Table 4. When the extreme conditions of segregation which causes honeycomb are known to exist in drilled cores, it is obvious that all lesser degrees of segregation also exist and it is but logical to conclude that the variability introduced in the laboratory specimens represented by Table 4 are present to an even greater degree in drilled cores.

Variabilities affecting the consolidation of the concrete and the distribution of the coarse aggregate in the mortar, regardless of how they are introduced, collectively result in nonuniformity of distribution of the coarse aggregate particles in the mortar. Practically all operations from the production of the coarse aggregate to the finishing of the pavement are included, but there are many causes of variability which cannot be included in this classification. Aggregates come from various sources and so do the cements. Imperfect control of the mixing water introduces variability. Conditions of exposure, curing, and climate, and undoubtedly many others, introduce variability. The effects of many of these causes, however, partially or completely disappear as the concrete approaches its ultimate strength.

Observation of both laboratory and field test results indicate that condition of exposure, curing, and climate has little or no effect upon the ultimate strength of concrete as long as the concrete has not been damaged during early ages by freezing or drying. Similarly, the difference in strength between gravel and crushed stone concrete, apparent at early ages, is of practically no importance at advanced ages, although this would not be true where gravel and crushed stone cannot be obtained under the same quality specification. Differences in strength due to the use of different brands of cement have also been observed to become greatly minimized as the concrete increased in age. As a matter of fact, some of the greatest nonuniformity in the 81 samples of cores was found in sections where only one brand of cement was used. Also, since the proportions for the mixtures are designed accurately for a given strength, taking into account the characteristics of the aggregates, and the control of the proportioning and the mixing water is in the hands of an engineer assigned to the work, no variability of large magni-

tude should result from this. Therefore, when the concrete attains the age when most cores are drilled, the effect of many of the causes of variability should have been greatly minimized.

The results of field beam tests from 1931 to 1941, inclusive, confirm this. The average of the standard deviations estimated for these years at the 3, 5, 7, and 14-day test ages are 117, 104, 100, and 98 pounds per square inch, respectively. Expressed in percentage of average strength, the values are 18.7, 14.4, 13.0, and 11.4, respectively. This is considerably different from what is found under laboratory conditions, where the absolute values evidently increase with age and the percentages tend to remain nearly constant. While accurate determinations could not be made at ages greater than 14 days, it is indicated that the absolute values will remain around 100 pounds per square inch at least up to 28 days and the percentages will decrease still further. All the variability in field concrete, except extreme segregation and curing, may be considered as being collectively represented in these values. The beam forms were filled by shoveling from batches deposited on the subgrade, preventing extreme segregation, and the beams were cured in moist sand.

These figures should carry considerable weight because of the great number of beams made each year, ranging from more than 17,000 in 1931 to about 3,000 in 1941, the number of beams being reduced as the methods of proportioning became established and as the work reduced in volume. They indicate that a large part of the variability present during the extreme early age of the concrete will disappear during the first month.

It was possible from these data to segregate the influence of the fact that the cement was obtained from different mills. The standard deviations of the average strengths obtained with the various brands of cement, as estimated at the 3, 5, 7, and 14-day test ages, were 43, 39, 33, and 33 pounds per square inch, or 7.0, 5.2, 4.4, and 3.9 per cent of the grand average strength. These values show practically the same trend, but they may not be taken as corresponding directly to the values given above, because they were not weighted in accordance with the number of beams tested from each cement, and because any data which did not include at least 25 beam tests were arbitrarily considered insufficient for accurate results and disregarded. The values shown are not indicative of all variability resulting from cements, since some variation may be expected to result from different shipments from the same mill, and are given only for illustrative purposes.

The standard deviations given were determined on the basis of beam strengths and not on the basis of individual breaks. Two breaks were

made on each beam and the average for these taken as the strength of the beam. The beam size was 6 x 6 x 30 in. and the test, while made in a so-called cantilever apparatus, was essentially a simple beam test over a 16-inch span loaded at the midpoint. It is regretted that no data can be presented for this type of specimen as to variability obtained under laboratory conditions.

It is hoped that this additional discussion will justify the contention that segregation is at least the major permanent cause of variability in pavement core strengths, and that the data in Table 4 "go a long way toward explaining the nonuniformity usually found in such data".

Mr. Jones seems to have arrived at the conclusion that pit run gravel was used in some of the mixtures represented by the core strengths cited. This is not the case. All coarse aggregates were processed to a definite specification, but they were not required to be delivered in separated sizes. He is undoubtedly right in his contention that the cylinder strengths need not be reported to the nearest pound. However, in this age of electrically driven, automatic calculators there exists no really good reason for rounding the values.

Mr. Wing calls attention to the possibility that the standard deviation, in the case of any given laboratory procedure, may be a constant percentage of the average strength. If this is true with a sufficient degree of accuracy, application of statistical methods in the analysis of test results will be greatly simplified. In tests performed by the Illinois Division of Highways on the class of concrete regularly used in highway construction, the only factor influencing the increase in strength being age, the minimum value of the standard deviation, expressed in percentage of average strength, seems to have occurred around the 14 or 28-day age. The amount of data available, however, does not justify a definite conclusion in this respect.

Table C is offered as evidence of the possibility that the standard deviation may be a definite percentage of the average strength for a given laboratory procedure. Each value given was determined from 20 sets of 6 tests with the proper correction for sample size. It is possible the variations shown are not significant, although as far as the average trend is concerned, the minimum standard deviation for the compressive specimens did occur for the 28-day age. The specimens were 2 x 4 in. mortar cylinders and 1½ x 1½ x 12 in. mortar beams. It is seen that the percentages for the flexural specimens are about two-thirds of those for the compressive specimens. It should be understood that although the size of the specimen in the case of the compressive test does not appear to have much effect on the value of the standard deviation, the

TABLE C—RELATION OF STANDARD DEVIATION WHEN EXPRESSED IN PERCENTAGE OF AVERAGE STRENGTH TO QUALITY AND AGE OF MORTAR

Age in Days	Gallons of Water per Bag of Cement				Av.
	7.5	6.3	5.4	4.6	
	2 x 4 in. Mortar Cylinders				
7	10.4	8.3	8.8	8.7	9.0
14	9.5	8.6	9.4	8.0	8.9
28	8.8	8.5	7.5	7.0	7.9
90	11.0	9.1	10.0	11.3	10.4
Av.	9.9	8.6	8.9	8.8	9.0
	1½ x 1½ x 12-in. Mortar Beams				
7	7.3	5.1	5.9	5.9	6.0
14	6.5	6.0	5.2	6.8	6.1
28	5.0	5.9	5.0	6.3	5.6
90	5.6	5.2	5.3	6.2	5.6
Av.	6.1	5.6	5.4	6.3	5.8

same may not be true in the case of the flexural test. The values for the small mortar beams in Table C, therefore, are not necessarily comparable with values obtained for 6 x 6 x 30-in. concrete beams.

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Design Diagrams for Square Concrete Columns Eccentrically Loaded in Two Directions*

By PAUL ANDERSEN†

SYNOPSIS

Building frames of reinforced concrete are continuous space structures. In the case of beam and slab construction the vertical and lateral loads are carried to square columns through a grid system of girders. Shears and bending moments are thus transmitted to the columns (generally square) in two perpendicular planes and can be resolved into single forces eccentrically placed in two directions with respect to the column axes.

In this paper are presented fifteen diagrams which will give directly, for square columns with symmetrical reinforcement and eccentrically loaded, the maximum compressive stress on the concrete as well as the maximum tensile stress in the reinforcement without first determining the position of the neutral axis. These diagrams can be used for sections reinforced with four, eight, twelve or sixteen bars.

NOMENCLATURE

A_s = cross-sectional area
a = length of side of square section
D = length of diagonal
d = distance from corner to reinforcing bar
e = eccentricity
f_c = maximum compressive stress on concrete
f_s = maximum tensile stress in reinforcing bars
f'_s = maximum tensile stress for equivalent section
M = bending moment
m = distance between adjacent corner bars
n = modular ratio
k = distance from apex of compression area to neutral axis, divided by length of diagonal
P = magnitude of load
p = steel ratio
x = distance between adjacent corner bars for equivalent section
Θ = angle between diagonal and line connecting center of section and point of application of load
C,T,c = coefficients

*Received by the Institute Aug. 14, 1940.
†Associate Professor of Structural Engineering, University of Minnesota, Minneapolis.

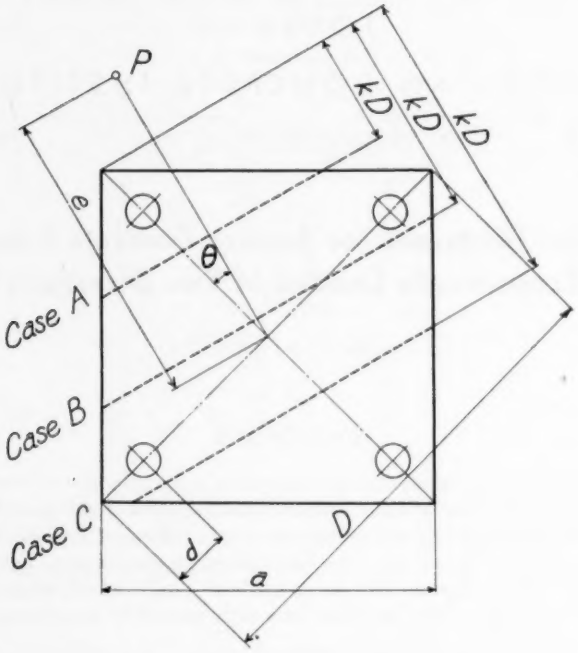


Fig. 1

REVIEW OF THEORY

Fig. 1 shows a square section reinforced with four rods and subjected to the action of a concentrated load applied at P. Depending on the position of the eccentric load the part of the section which will sustain compressive stresses may form a triangle (Case A), a trapezoid (Case B) or a pentagon (Case C). If the conventional assumptions of reinforced concrete design are adopted and supplemented by the supposition of perpendicularly of neutral axis to plane of bending, equations can be set up from which the position of the neutral axis can be determined.* The usual assumptions of reinforced concrete practice are preservation of plane sections, proportionality between stress and strain, and disregard of tensile stresses in concrete. A study of the additional assumption (neutral axis normal to plane of bending) has been made recently.† This study showed that the error involved in this last assumption is extremely small.

The equations from which the position of the neutral axis can be determined are:

*See *Civil Engineering*, Oct. 1939, pp. 616-618.
†See *Civil Engineering*, Jan. 1940, pp. 37-39.

Case A:

$$k^4 + (2\frac{e}{D} - cos\theta)k^3 + 3np\,\frac{e}{D}kcos2\theta - \frac{3}{2}npcos2\theta\left[\frac{e}{D}cos\theta + (\frac{1}{2} - \frac{d}{D})^2\right] = 0 \quad \ldots\ldots\ldots\ldots (1)$$

Case B:

$$k^3 + (3\frac{e}{D} - \frac{3}{2}cos\theta)k^2 - \frac{cos\theta - sin\theta}{4}(6\frac{e}{D} - 2cos\theta - sin\theta)k + 3np\frac{e}{D}(cos\theta + sin\theta)k - \frac{3}{2}(cos\theta + sin\theta)\left[\frac{e}{D}cos\theta + (\frac{1}{2} - \frac{d}{D})^2\right] + \frac{(cos\theta - sin\theta)^2}{16}(4\frac{e}{D} - cos\theta - sin\theta) = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Case C:

$$k^4 + (2\frac{e}{D} - 3cos\theta)k^3 + 3cos\theta(cos\theta - 2\frac{e}{D})k^2 - \left[cos^3\theta + 3\frac{e}{D}(npcos2\theta - 1)\right]k + \frac{e}{2D}cos\theta(1 + 2\,sin^2\theta) + \frac{cos2\theta}{8} + \frac{3}{2}\,np\,cos2\theta\left[\frac{e}{D}\,cos\theta + (\frac{1}{2} - \frac{d}{D})^2\right] = 0 \quad \ldots\ldots\ldots\ldots\ldots (3)$$

With k known the maximum compressive stress, f_c, on the concrete can be determined by:

$$f_c = \frac{P}{Ca^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Where

$$C = \frac{c_1 + 3np\,(2k - cos\theta)\,cos2\theta}{6kcos2\theta} \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

In this expression c_1 equals for the three cases of stress distribution.

Case A: $4k^3$

Case B: $6k^2(cos\theta - sin\theta) - 3k\,(cos\theta - sin\theta)^2 - \frac{(cos\theta - sin\theta)^3}{2}$

Case C: $-4k^3 + 12k^2cos\theta - 6k + cos\theta\,(1 + 2\,sin^2\theta) \quad \ldots\ldots\ldots\ldots (6)$

In all three cases the maximum stress in the tensile reinforcement is

$$f_s = nTf_c \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

Where

$$T = (1 - \frac{d}{D}) \frac{cos\Theta}{k} - 1 \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

DIAGRAMS FOR USE IN DESIGNING

One of the most frequently occurring problems in the design of reinforced concrete columns is after shears and moments have been determined to find the maximum stresses in the concrete and the steel.

The first step is to substitute a single force whose magnitude and location are readily determined from the given shears and moments. In this paper the point of application of the single force is expressed by the eccentricity, e, and the angle of eccentricity, Θ, as indicated in Fig. 1.

In order to compute the maximum stresses by equations (4) and (7) the position of the neutral axis must first be determined from equations (1), (2) and (3). The diagrams presented in this paper eliminate the necessity of computing k, the stress coefficients C and T are given as functions of the position of the load and the dimensions of the section.

The diagrams give directly the stress coefficients C and T, as defined by equations (5) and (8) respectively, for sections reinforced with four rods. Three embedment ratios, d/D, have been assumed, namely 0.1 (Diagrams 1-5), 0.15 (Diagrams 6-10) and 0.2 (Diagrams 11-15). Each assumed embedment ratio has been plotted for five angles of eccentricity, namely 0 degrees (two equal bending moments), 10, 20, 30 and 40 degrees. Finally different steel percentages and modular ratios have been taken into account by varying the product pn from 0.1 to 1.0.

SECTIONS REINFORCED WITH EIGHT BARS

Diagrams 1-15, which are plotted for sections with four bars can be used for members reinforced with eight bars by making use of an equivalent four-bar section as explained in the following:

In Fig. 2 is shown a square section reinforced with eight bars, each with a cross-sectional area of A_s. The equivalent four-bar section is the square section of the same size reinforced with four bars whose total cross-sectional area equals the total area of the original reinforcement but so placed that for a given load the concrete stress intensities of the two sections will remain identical.

As the reinforcing bars are equally effective in tension and compression, it is readily seen that for a given neutral axis, the sum of all internal moments will remain unchanged, if the reinforcement is concentrated at such four points that the moments of inertia with respect to the two axes 1-1 and 2-2 for the four equivalent bars are the same as for the original eight bars. Referring to Fig. 2 this requirement is fulfilled if

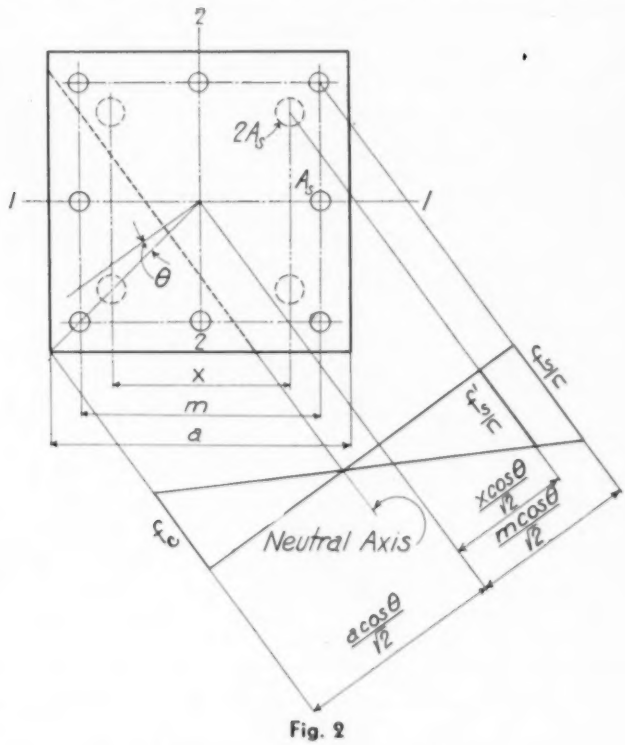


Fig. 2

$$6 \times A_s \times \left(\frac{m}{2}\right)^2 = 4 \times 2A_s \times \left(\frac{x}{2}\right)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

$$x = 0.866m \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

If, therefore, for a given section with eight bars, it is desired to find the maximum concrete stress, the equivalent four-bar section as determined by equation (10) can be analyzed by equations (1) to (6) or by using the corresponding diagrams. The concrete stress, f_c, thus found will be identical to the concrete stress in the original eight bar section; but the tensile stress, f'_s in the most highly stressed of the four bars will be smaller than the desired tensile stress, f_s, in the original eight bar section. A relation between these two stresses can be derived from similar triangles in Fig. 2, thus:

$$\frac{f_c + \frac{f_s}{n}}{a + m} = \frac{\frac{f_s}{n} - \frac{f'_s}{n}}{m - x} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

$$f_s = \frac{nf_c\,(m - x) + f'_s\,(m + a)}{a + x} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

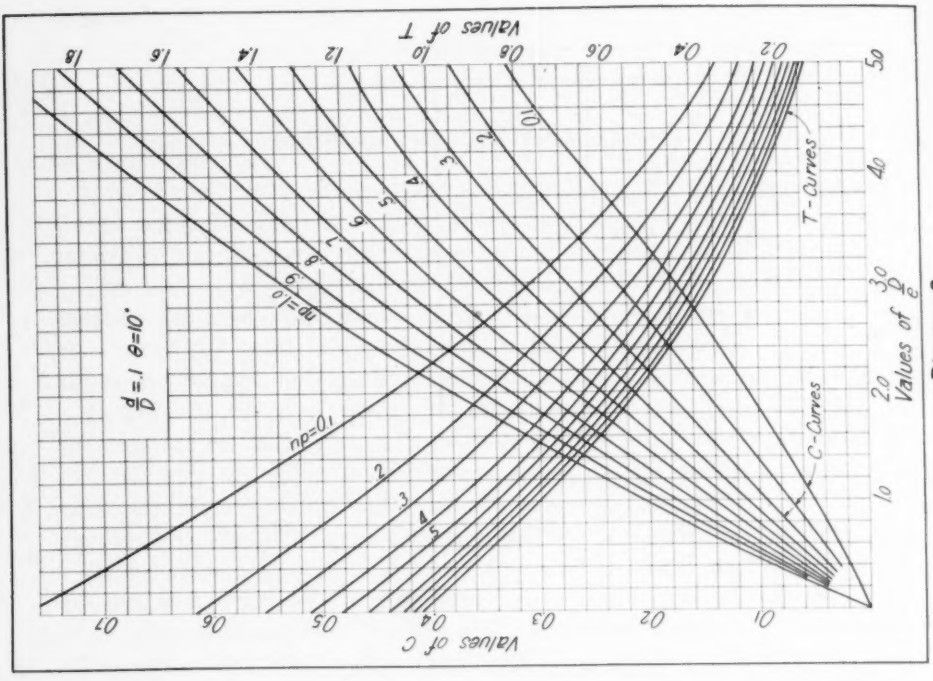


Diagram 2

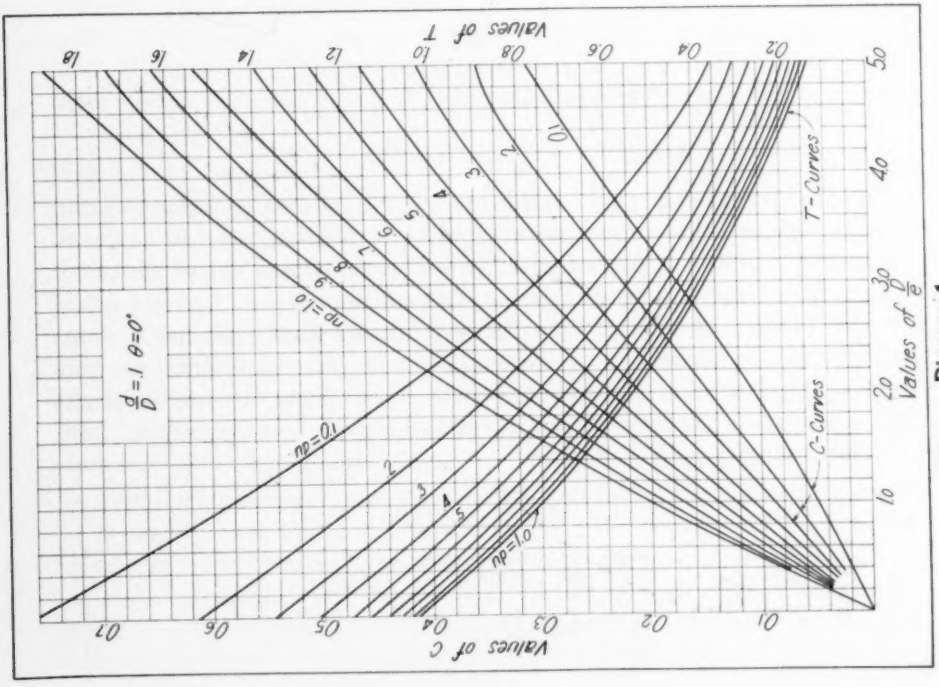


Diagram 1

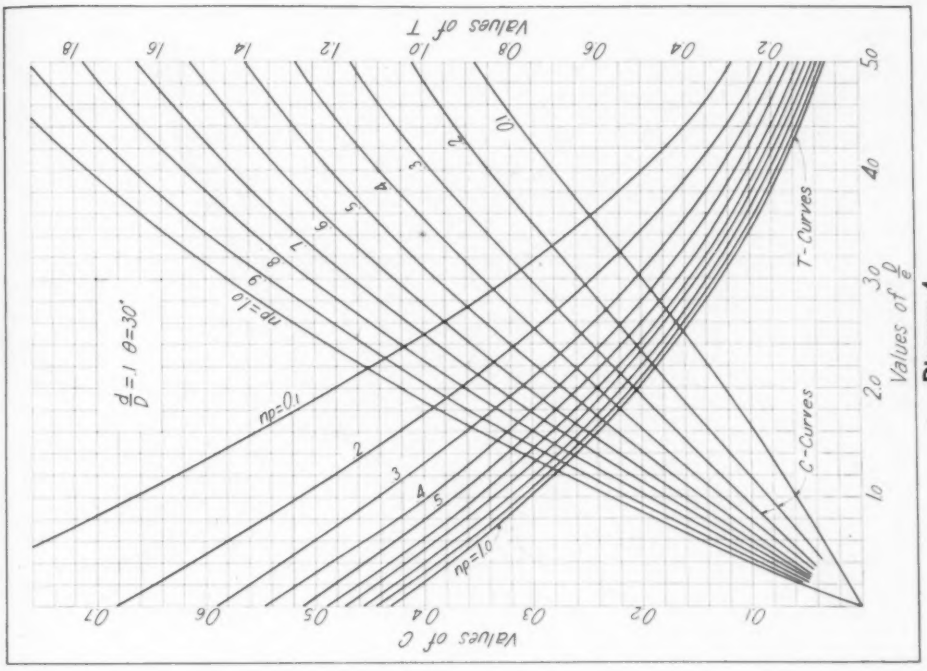


Diagram 4

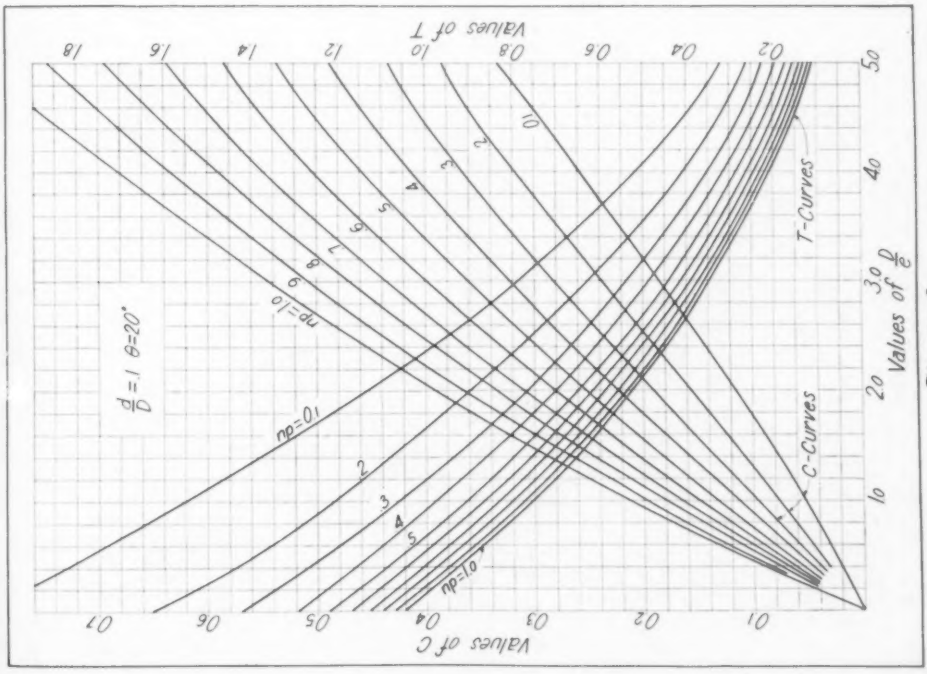


Diagram 3

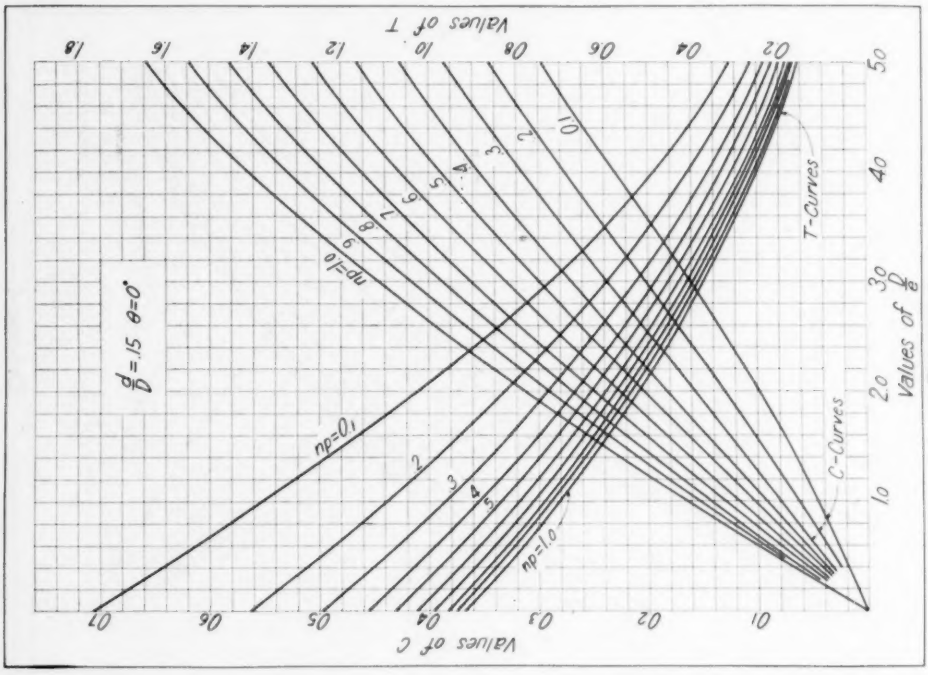


Diagram 6

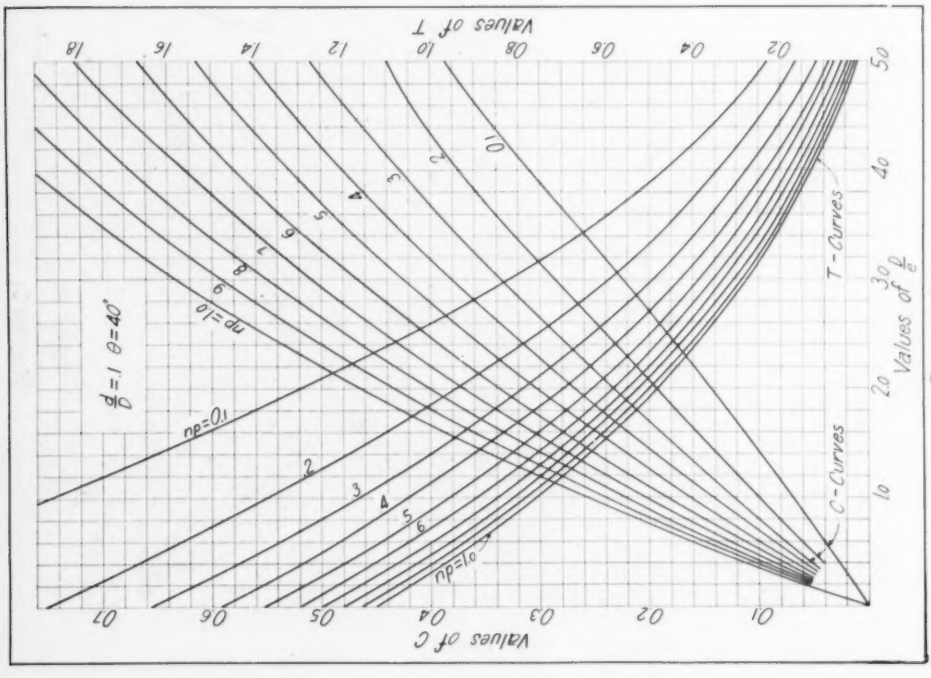


Diagram 5

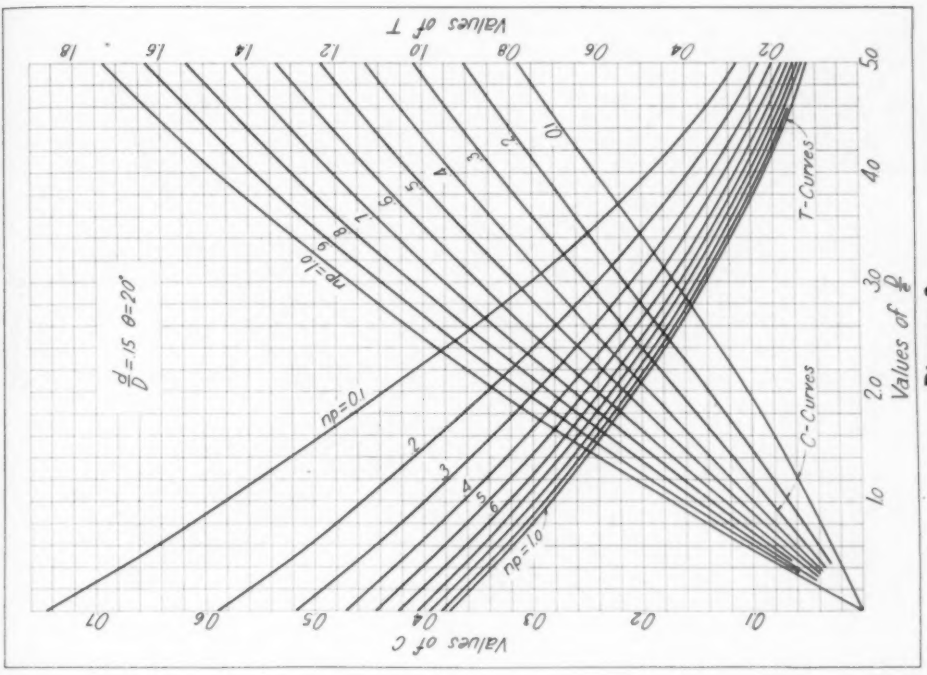


Diagram 8

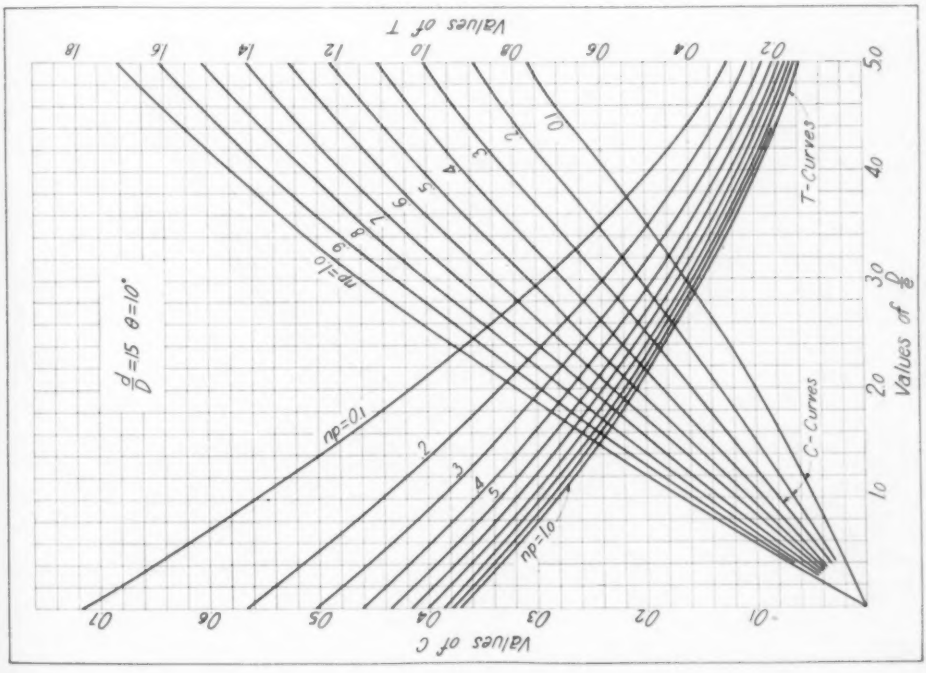


Diagram 7

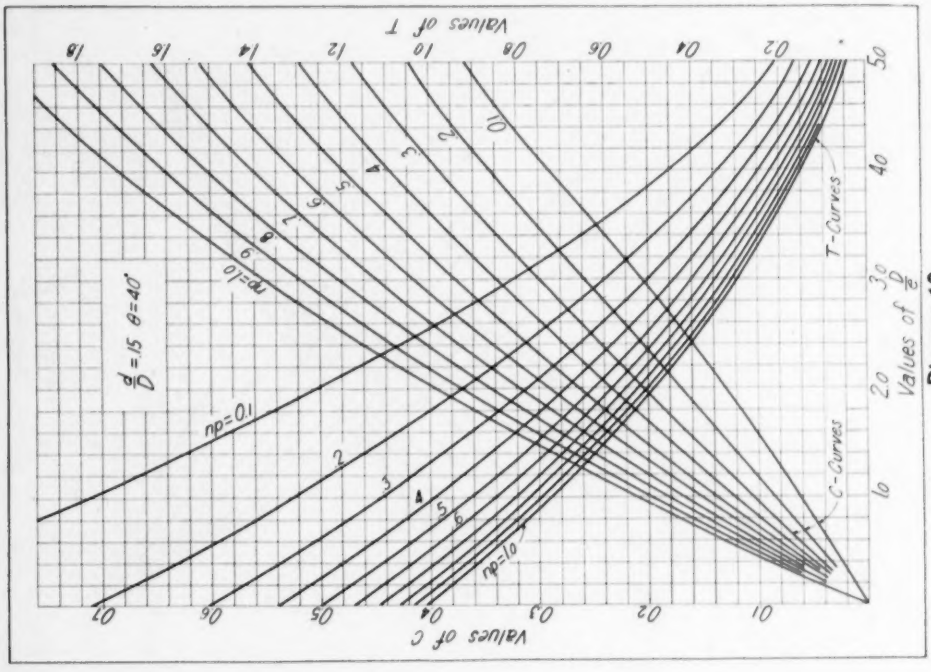


Diagram 10

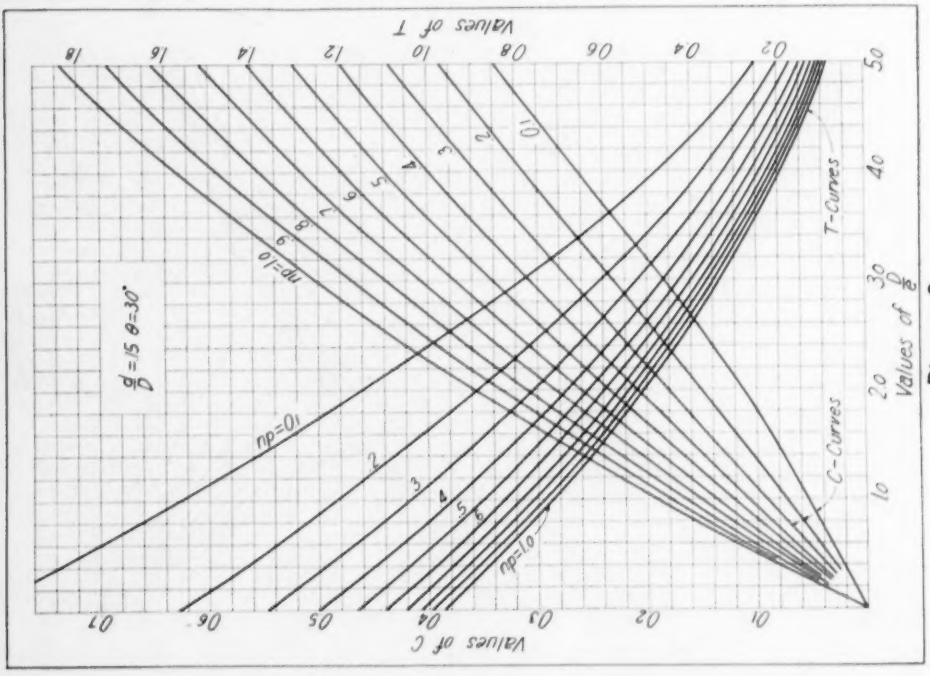


Diagram 9

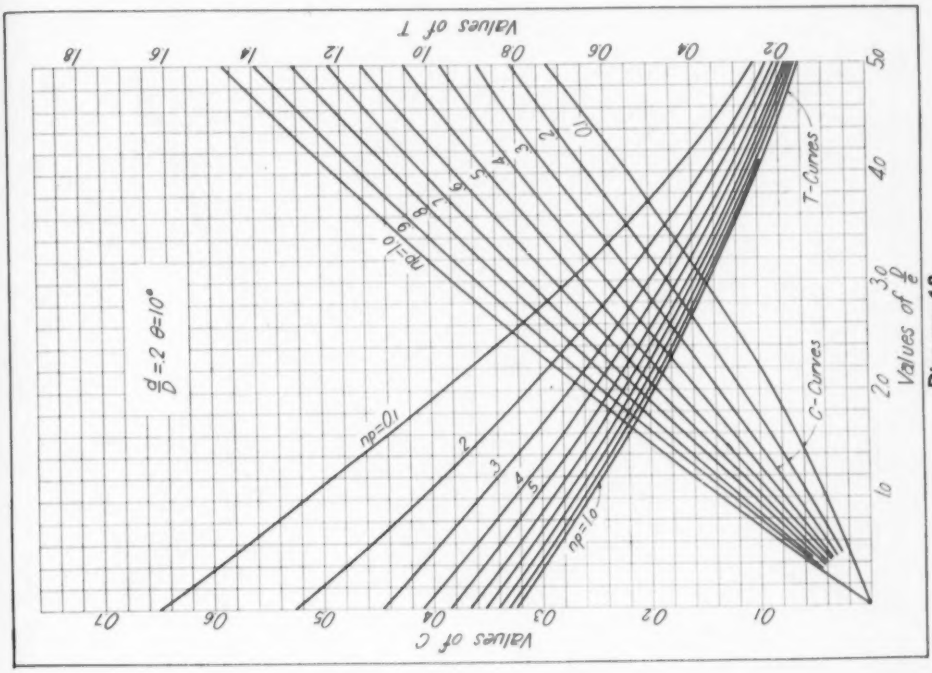


Diagram 12

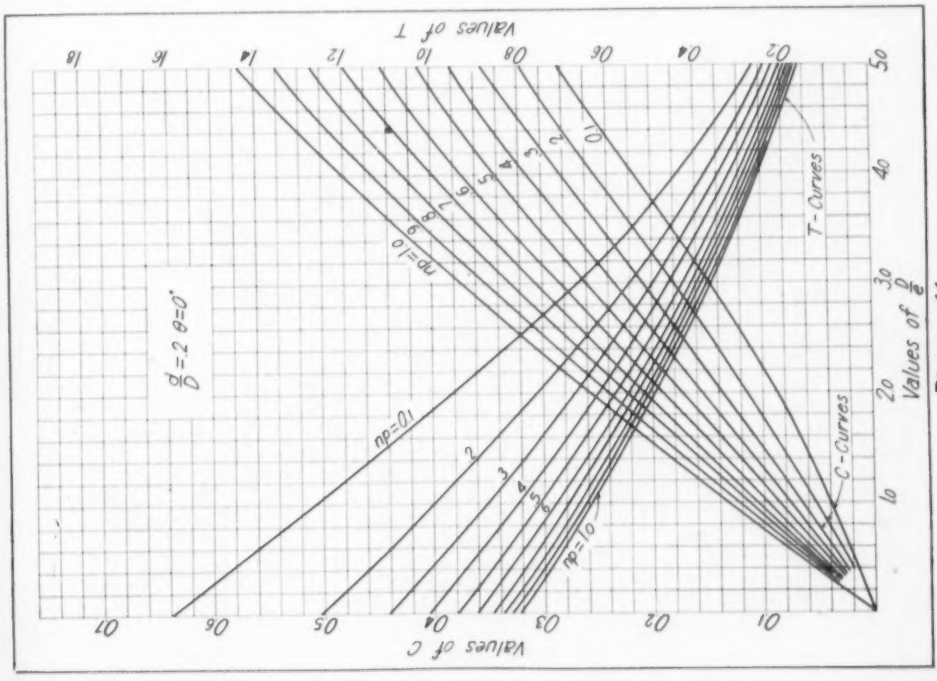


Diagram 11

Diagram 9

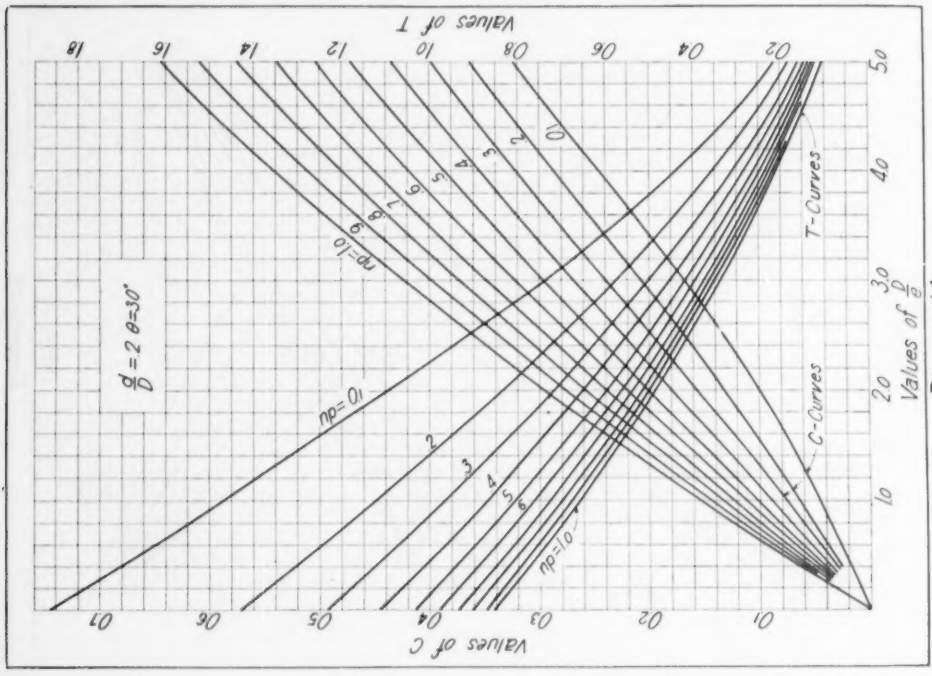


Diagram 14

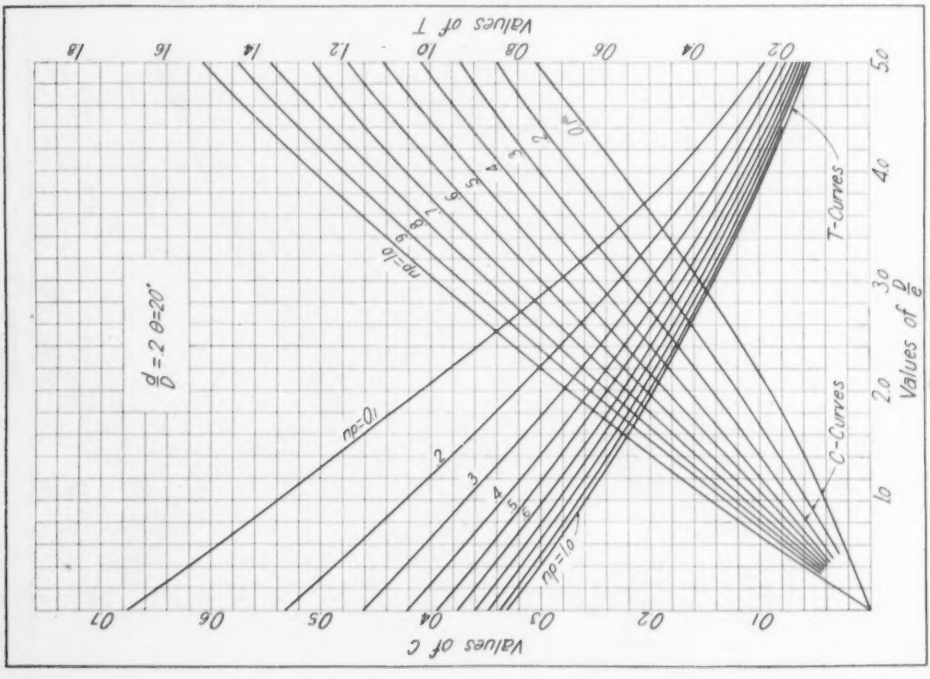
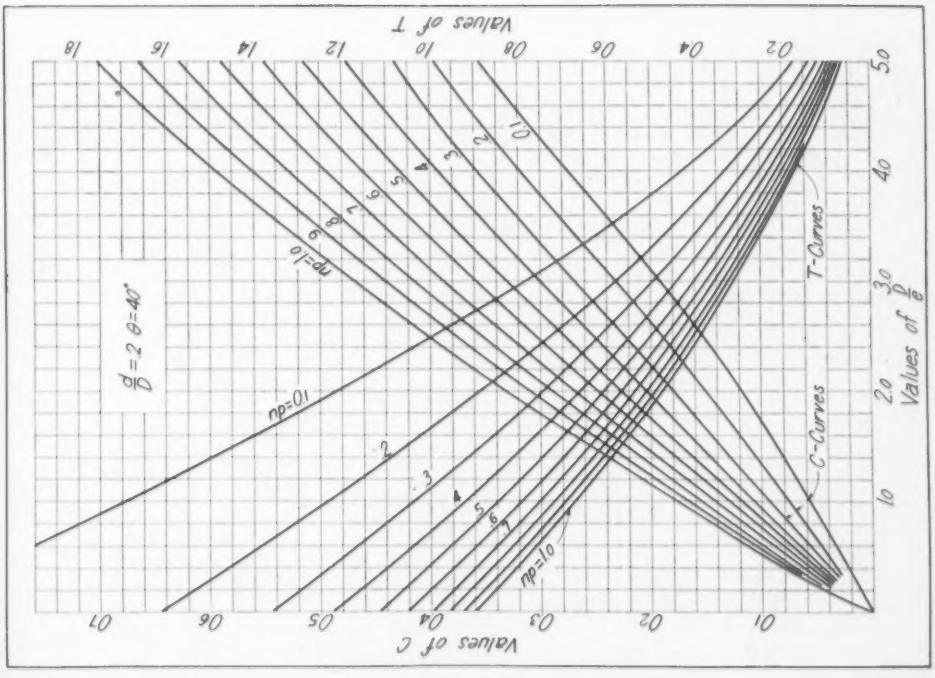


Diagram 13

Diagram 15


Values of T
Values of C
Values of $\frac{D}{e}$
T-Curves
C-Curves
$\frac{d}{D}=2\ \theta=40°$
np=10
np=1.0

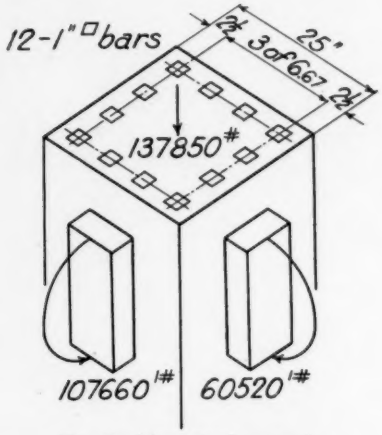


Fig. 3—Numerical example

SECTIONS REINFORCED WITH TWELVE BARS

Columns having twelve bars can be treated similar to columns with eight bars. The corresponding relations become:

$$8 \times A_s \times \left(\frac{m}{2}\right)^2 + 4 \times A_s \times \left(\frac{m}{6}\right)^2 = 4 \times 3A_s \times \left(\frac{x}{2}\right)^2 \quad \ldots (9a)$$

$$x = 0.839m \quad \ldots (10a)$$

Similar to the preceding case the concrete stress in the equivalent section will be the desired stress and the correct steel stress can be found from equation (12).

NUMERICAL EXAMPLE

Fig. 3 shows a square concrete column reinforced with 12 one-inch square bars and subjected to the action of a direct load and two bending moments acting in perpendicular planes. The eccentricity and angle of eccentricity of the equivalent single load are:

$$e = \frac{\sqrt{107660^2 + 60520^2}}{137850} \times 12 = 10.75 \text{ in.}$$

$$tan\ (45° - \theta) = \frac{60520}{107660}; \theta = 15° 40'$$

The distance between corner bars of the equivalent four-bar section is:

$$x = 0.839 \times 20 = 16.78 \text{ in.}$$

The embedment ratio for the equivalent section is:

$$\frac{d}{D} = \frac{4.11}{25} = 0.164$$

If 10 is assumed for the modular ratio, then:

$$pn = \frac{12}{625} \times 10 = 0.192$$

An inspection of Diagrams 7, 8, 12 and 13 gives:

Diagram 7; $d/D = 0.15$; $\theta = 10°$; $C = 0.22$; $T = 0.55$
Diagram 8; $d/D = 0.15$; $\theta = 20°$; $C = 0.23$; $T = 0.56$
Diagram 12; $d/D = 0.2$; $\theta = 10°$; $C = 0.20$; $T = 0.51$
Diagram 13; $d/D = 0.2$; $\theta = 20°$; $C = 0.21$; $T = 0.52$

Interpolating for an embedment ratio of 0.164

$$C = 0.225 + (0.205 - 0.225)\frac{0.164 - 0.15}{0.2 - 0.15} = 0.22$$

$$T = 0.555 + (0.515 - 0.555)\frac{0.164 - 0.15}{0.2 - 0.15} = 0.544$$

According to equation (4) the maximum compressive stress on the concrete is:

$$f_c = \frac{137850}{0.22 \times 625} = 1003 \text{ lb. per sq. in.}$$

and according to equation (8) the maximum tensile stress in the equivalent four-bar section

$$f'_s = 10 \times 1003 \times 0.544 = 5440 \text{ lb. per sq. in.}$$

The correct maximum tensile stress for the twelve-bar section is, according to equation (12):

$$f_s = \frac{10 \times 1003 \times (20 - 16.78) + 5440 \times (20 + 25)}{25 + 16.78}$$

$$f_s = 6632 \text{ lb. per sq. in.}$$

ACKNOWLEDGMENT

The computations and drafting work necessary for the production of the fifteen diagrams presented in this paper were done with the assistance of personnel furnished by the Works Progress Administration, Official Project No. 665-71-3-69, Sub-Project No. 303.

Discussion of this paper should reach the ACI Secretary in triplicate by Mar. 1. 1942, for publication in the JOURNAL for June, 1942

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Vol. 13 No. 6 7400 SECOND BOULEVARD, DETROIT, MICHIGAN June 1942

Discussion of a paper by Paul Andersen:

Design Diagrams for Square Concrete Columns Eccentrically Loaded in Two Directions*

DISCUSSION By K. HAJNAL-KONYI†

I should like to mention another solution of this problem, given by Professor Roussopoulos in *Annales Techniques* (Athens) of May 1 and 15, 1933, and also published in *Beton u. Eisen* 1939, No. 5, pp. 79 to 87.

Professor Roussopoulos' solution is more general since it refers to a rectangular cross section and can be applied both for eccentric tension and compression. An equation is given to determine the limit between "Case C" in Doctor Andersen's paper and between the case when the neutral axis does not intersect the cross section, i.e. the case of small eccentricities, when the load acts within the "core" of the cross section and the stresses can be calculated directly without determining the neutral axis from a complicated operation.

*ACI JOURNAL, Nov. 1941; *Proceedings* V. 38, p. 149.
†London, England.

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Some Experience With Prestressed Steel in Small Concrete Units*

By R. E. DILL†
Member American Concrete Institute

SYNOPSIS

Preventing early bond by using greased or oiled round steel rods and slowly stressing these rods against the body of the concrete, the writer built some 1400 posts and several hundred building slabs. With steel in tension and concrete in compression, neither posts nor slabs cracked and so far show no weathering. A laboratory test shows how a beam so built acts when loaded to failure and the curves show how high strength steel may be used to save steel or to produce higher strength beams without increasing size or weight.

This paper has to do with one method of using prestressed steel in fence posts and in small building slabs all manufactured as precast units and presents data furnished by a laboratory test of two beams identical except that in one beam the reinforcing was thus prestressed and in the other it was not.

There are two ways to prestress the steel in concrete. One way is to stress the steel and pour the plastic concrete around it holding the steel in a stressed state until the concrete has hardened. It is not often practical to hold the steel in its stressed condition until the concrete has fully hardened. The plastic flow of concrete is considerable when it is green. Since loss of stress in the steel due to plastic flow, shrinkage from drying, and the compressive shortening of the concrete due to the steel stress cannot be compensated for when this method is used, its usefulness is limited.

The second method is to coat the steel with a lubricant to prevent early bond of the concrete to the steel. As the concrete hardens, stress

*Received by the Institute, Jan. 22, 1941.
†Alexandria, Neb.

the steel against the body of the concrete, continuing the tightening until the desired stress is reached and held.

Using this method the writer built about 1400 fence posts in 1928 and 1929. Each post was 2¼ in. square at the top, 6 ft. long and 4¼ in. square at the bottom end. The reinforcing was one scant ⅜ in. round rod with rolled threads on one end. The threaded end was allowed to protrude 1¼ in. from the top of the post. All the steel except a hook at the bottom was coated with asphalt, grease or form oil. The day following the pouring of the concrete a 2 in. cast iron washer was slipped over the protruding end of the reinforcing, a nut screwed on tightly enough to produce some stress in the steel and some compression in the concrete, taking care not to crush the green concrete. The posts were then removed from the forms and the tightening process was continued as convenience offered or necessity required.

The use of "scant" ⅜ in. steel, that is, steel that does not measure "full", is required if standard nuts are used as the rolling process raises threads that are in part above the original surface. If full ⅜ in. steel rods are used the rolling process raises threads which require oversize nuts. Either combination is satisfactory.*

The use of rolled threads rather than cut threads is necessary because the cutting process reduces the net area of the rod so greatly as to pretty much nullify the gains to be derived from prestressing. The net area of the threaded section is reduced but very little when rolled threads are used.

These 12 and 13 year old posts have shown almost no deterioration. There is very little transverse cracking and none which cannot be reasonably attributed to some over load. There is no longitudinal cracking except at the top of some posts where the partly hardened concrete was subjected to overload from too great early tightening. All the rods were retightened a year or more after the posts were made.

After satisfactorily solving the application of this method to fence posts the writer attemped to apply it to building slabs. After several failures the satisfactory design herewith presented was evolved. Two lengths, 12 ft. and 14 ft., respectively, were built and used. To conserve space only one will be described in detail, the shorter one. Sheet steel twelve feet long was formed so that it was two feet wide and had depressions at the side as shown in the cross section Fig. 1.

The forms were modified at the ends to carry the depressions (shown at the sides) around the ends, the rib or flange continuing around ends as well as the sides of the slab. These end ribs were reinforced with a bonded 10 gauge wire. The longitudinal reinforcing used was scant

*Buffalo Bolt Co., and the Bethlehem Steel Co., furnish such material with rolled threads.

3/8 in. rods, one in each longitudinal rib, greased or oiled, as in the posts, but with rolled threaded ends protruding from both ends of the slabs. No other reinforcing was used and none found necessary. There are no shrinkage and no temperature cracks in either direction. The design called for a web thickness of one inch. Some by accident were made with a web thickness of only 5/8 in. and have been used without failure ever since. Such thin slabs, however, were found to be difficult to handle without damaging them. In the 14-ft. slabs the rib was made deeper and wider and scant 1/2-in. instead of scant 3/8-in. steel was used.

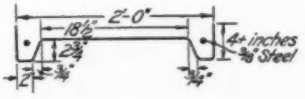

Fig. 1

The position of the reinforcing between the slab ends should be determined by the position in which the slab is to be placed in use. If to be placed vertically the steel should be about the middle of the rib (Fig. 1). If the slab is to be used horizontally as ceiling or roof, (the ribs of course should be on its under side) the reinforcing should be very near the bottom of the rib at the mid point. Its end position should be unchanged or slightly higher.

In 1929 the writer built a single-walled garage using these 12-ft. slabs for walls and roof. In 1935 and 1936 he built a double walled house approximately 42 by 32 ft. using 14-ft. slabs for roof and part of the ceiling and 12-ft. slabs for the rest of the ceiling, for inner walls, partitions and outer walls. The single walled garage is cold in winter, hot in summer. The double walled house, with the space between inner and outer walls filled with straw, is so fully insulated that it is cool in summer and is successfully heated throughout in winter with a specially designed but open fireplace at small cost. There is as yet no observable deterioration of the slabs.

Some years ago at the University of Nebraska two beams were built duplicate in all respects except that in one beam the two reinforcing rods of ordinary* quality were bonded and in the other they were stressed in tension against the body of the concrete. They were then identically loaded to failure. Fig. 2 shows the load-deflection curves for the two beams. At the first crack, which occurred at almost the same deflection in each beam, the prestressed beam was supporting a load of 7500 lb. as against a load of 4500 lb. for the beam with the steel not prestressed.

*Elastic limit, 40,000 psi.; tensile strength, 60,000 psi.

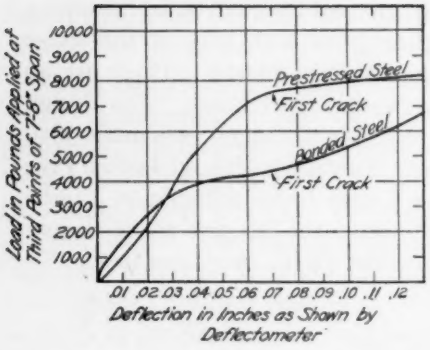

Fig. 2—Load deflection curves for two beams

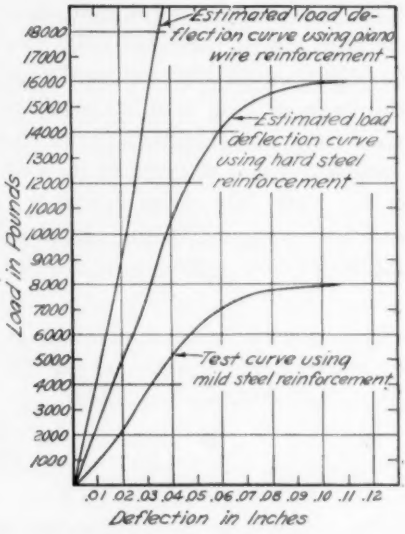

Fig. 3—Load deflection curves estimated and form test

By prestressing the additional strength of high strength steel may be used. In Fig. 3 the lowest line duplicates the prestressed beam curve of Fig. 2. The middle curve indicates the estimated load-deflection curve of a beam with steel of twice the tensile strength of that used in the first beam, doubly stressed; and the top-most curve the estimated load-deflection curve of a steel four times as strong as that used in the first beam, quadruply stressed. (Each of the latter curves is slightly in excess of actual load carrying capacities since to balance the greater tension in the steel at the bottom of these slabs there must be employed a greater depth of the concrete in their top sections to furnish the equalizing compression. This requirement reduces the length of the lever arm from the center of tension in the steel to the center of compression in the concrete.)

A unique value of prestressing is in connection with the occasional need to avoid any sag in a beam. For instance in a roof slab any sag whatever may be avoided or a decided arch may be secured by placing the mid-section of the reinforcing in the lower third of the slab and subsequently prestressing the reinforcing. If the concrete is not past the stage of plastic flow the slab will arch and do so without any cracking.

Discussion of this paper should reach the ACI Secretary in triplicate by Mar. 1, 1942, for publication in the JOURNAL for June, 1942

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Concrete Specifications and Water Content of Concrete*

By F. J. WARBERG†

SYNOPSIS

A plea for specifying definite credits, accruing to the owner for every failure of his contractor to meet specification requirements, heavy enough to make non compliance unprofitable; a plea to owners that a competent, honest inspector is well worth his hire, and suggesting a test for water content of concrete after it has set. A testing laboratory should not be a subcontractor.

Experienced concrete engineers and designers are well aware that, no matter how carefully a design is worked out, it can still be wrecked or nullified by vague specifications, or lack of competent field inspection.

The construction engineer on a concrete job nowadays has most of his worries over controling the water content of his concrete.

Many devices are on the market but none of them foolproof. In 30 years experience I have not seen any that could not be circumvented by any resourceful contractor. Why does a contractor want to pour "soup"? Simply because it is more profitable to him! That in general is the reason for most of the objectionable features of construction work.

Constant, honest, intelligent and efficient inspection by a competent man is still the only real way to get good water control on a concrete job.

One would almost think that honest, competent men are scarce, or too high-priced, for the practice prevails among designers and specification writers of putting small trust in competent honest inspection. Instead they specify a generous excess of cement and hope to get the strength required in spite of all that may happen, or not happen. After all, the architect, or engineer, has to protect the owner against a poor job. He knows that poor concrete makes for excessive maintenance costs and robs the owner of the flexibility in load carrying capacity so sorely needed in these days of ever increasing load carrying demands.

So, instead of demanding 3000 lb. concrete with 5 bags of cement per cu. yd., we specify 6 bags per cu. yd. What this means on a good-sized

*Received by the Institute Aug. 28, 1940.
†Registered Structural Engineer, Dearborn, Mich.

job of, say 8000 cu. yds. is easily seen. If cement is selling at $2.40 per bbl. this means $4800.00 added to the cost. It should not take more than the 6 warm months to pour such yardage, so a competent inspector could be hired at $300 per month and $3000 cut off the cost of the job.

Another bad practice: The cost of properly designing a reinforced concrete job to satisfy any up-to-date code is considerably higher than for a structural steel job; specifications are therefore written to relieve the architect of certain field inspection expenses such as tests and test cylinders, and the contract so drawn as to make the contractor engage and pay for the services of an "approved" testing laboratory.

To my way of thinking these expenses should properly be borne by the owner directly (incidentally saving him the contractor's profit on this item), because otherwise it puts the testing laboratory in a poor light if something is not as it should be; the laboratory in the position of "biting the hand that feeds it"—not conducive to candid reports.

Another common failing of concrete specifications is failure to provide definite penalties for deviation from specified requirements. If the contractor is required to make screen tests of the aggregate or slump tests of the concrete at certain intervals and omits to do so, certain definite penalties should be specified. If he is required to put six bags of cement into every yard of concrete and fails to do so, penalties should be exacted. If he fails to provide the required strength, uses excess water, definite penalties should be provided. (This last item, of course, is one of the most important ones and one of those hardest to enforce, the slump test being what it is.) Unless penalties are large enough to make bad practice unprofitable, they cause friction but accomplish no good.

The placing of reinforcement could be handled better in many specifications. In a modern building it is next to impossible to prevent workmen from walking over steel already in place. The plumbers, steamfitters, electricians and ventilating people must place their sleeves, inserts, conduits, etc.; runways for buggies must be laid. If an electrician gets his sleeves in first where a reinforcing bar should go the chances are that the bar will be placed lower than it should be; if he doesn't, the chances are that he will force the reinforcing bar out of his way. Penalties should apply. Displacement of steel by walking over it should be guarded against by providing chair bars and chairs often enough and substantial enough to keep the steel in place even when a 200 pound man carrying a 100 pound load steps on it anywhere with impact (he will, you know), or else the job should be gone over again carefully just before concreting begins, which means that the inspector definitely must be clothed with power, backed up with penalties, to forbid or stop concreting.

On one recent job I had the stirrups come shopwelded into one unit for each end of the beam. I liked that—no chance for displacement.

Concrete curing procedures are often hard to enforce. The poured deck is so handy for carrying on preparations for extension of the pour and for storing the required materials, that men are constantly walking over it, and they object to getting their feet wet!

Definite penalties should be provided for failure to keep the concrete wet after initial hardening.

Cylinders molded on the job but cured in the laboratory are only indicative of what it is possible to get *if job curing were perfect.* How good is the concrete in the job? So I like my cylinders stored on the job under conditions which, as nearly as possible, approximate the condition of the concrete in the job. I like to go farther, I like to select a 12-in. wide beam form, with no concentration in midspan, and set here, horizontally, a fibre test cylinder mold so cut (Fig. 1) that it can be filled and then closed on top. When the forms are stripped this will be comparatively easy to remove and it is truly representative of the job concrete.

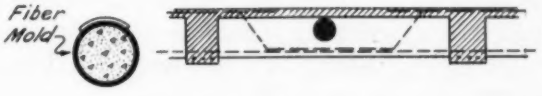

Fig. 1

There are two good reasons for demanding that the report show test cylinder weights before testing, to the nearest ¼ oz.: (1) if there is honeycomb in the interior, it will definitely show up in the weight, provided the cylinder is of required size; (2) the weight of concrete per cubic foot is a direct indication of the amount of mixing water used in that concrete, and for any given mixing proportions of cement and aggregate a definite curve for the relationship between the weight of the test cylinder and the quantity of mixing water used can be drawn. Incidentally it is, of course, a straight line as a little thought will show.

This is reliable only when both the weight and the volume of the test cylinder are known to close enough limits so that the weight of the concrete per cu. ft. can be calculated to the nearest ¼ lb. To illustrate:

Assume a mix of 6 bags cement, 1550 lb. of sand, 1830 lb. gravel and water to be varied for purposes of illustration. Assume sand and gravel to be measured by weight damp and loose and that the sand contains 62 lb. of water (4.1 per cent) and the gravel 25 lb. of water (1.35 per cent) also that specific gravity of sand and gravel is 2.64 or density = 165 lb. per cu. ft. Then for 6 gal. of water per bag of cement:

sand	1550 − 52 =	1488 lb. =	9.03	cu. ft.
gravel	1830 − 25 =	1805 lb. =	10.96	cu. ft.
cement 6 bags at 94 lb.	=	564 lb. =	2.94	cu. ft.
water	36 gal. =	300.6 lb. =	4.81	cu. ft.
Totals for batch		= 4157.6 lb. =	27.74	cu. ft.

$$\text{Wt. of concrete} = \frac{4157.6}{27.74} = 149.8 \text{ lb. per cu. ft.}$$

Volume of 6-in. diam. cylinder 12 in. high = 0.1965 cu. ft.
Wt. of cylinder: = 149.8 + 0.1965 = 29.45 lb.

With 8 gal. of water per bag of cement:

sand	1550 − 62 =	1488 lb. =	9.03	cu. ft.
gravel	1830 − 25 =	1805 lb. =	10.96	cu. ft.
cement 6 bags at 94 lb.	=	564 lb. =	2.94	cu. ft.
water	48 gal. =	400.8 lb. =	6.42	cu. ft.
Total for batch	=	4257.8 lb. =	29.35	cu. ft.

Wt. of concrete $= \frac{4257.8}{29.35} = 145.1$ lb. per cu. ft.

Wt. of test cylinder = 0.1965 + 145.1 = 28.55 lbs.

It is obvious that if material of a smaller specific gravity is added to a mixture of materials having higher specific gravity, the overall density of the resulting mix will be lowered in direct proportion to the amount of lighter material added hence, a straight line relationship between weight of cylinder and water content. Test cylinders for purpose of checking water content only may be molded and weighed as soon as set up enough to remove mold. Of course they should be accurately molded or else weighed in water also to determine actual volume. For the case illustrated the relationship is as shown in Fig. 2.

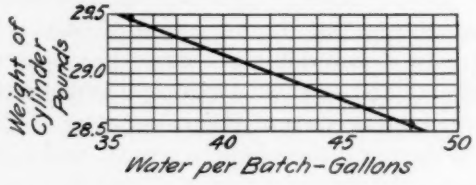

Fig. 2

The point may be made that there is no way to tell which part of the deficiency is due to excess water and which is due to hidden honeycomb. The answer is, of course, to be careful in making the cylinders, to rod them properly; then no concealed honeycomb need be feared, and observable honeycomb or faulty volume can be corrected by weighing the cylinder in water also.

In conclusion, since we seem to have the means to arrange contracts and specifications so that contractors do not profit by poor work, we should use those means and take full advantage of it in design. A little owner-education along these lines would, of course, be a great help.

Discussion of this paper should reach the ACI Secretary in triplicate by Mar. 1, 1942, for publication in the JOURNAL for June, 1942

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Proposed Recommended Practice for use of Metal Supports for Reinforcement*

Reported by ACI Committee 319 (formerly 507)

WILLIAM F. ZABRISKIE
Chairman

CARL S. BARRY
D. H. BITNEY
MARTIN STELJES
W. E. WHITE

1. General

All reinforcing steel shall be accurately located in the forms and held firmly in place before and during the placing of concrete, by means of metallic supports, spacer bars or wires.

2. Supports

Bar supports shall be sufficiently strong and stable to sustain construction loads without permanent distortion or displacement. (see "Types of Devices") These shall be designed to permit full embedment in the concrete without the formation of voids and with a minimum exposure of metal on finished surfaces. They shall, in combination with tie wires or clips, provide positive means for establishing and maintaining operative association with the reinforcing steel.

3. Number and location of supports

Minimum requirements for number and location of supports and spacers under various usual conditions shall be as set forth in Tables 1 to 4.

4. Responsibility

The number, type, and location of supporting and spacing devices shall be clearly indicated on the drawings which show placing of reinforcement.

It shall be the responsibility of the seller of such devices to furnish the quantities, types, and capacities as indicated on the drawings, with the necessary identification marks.

It shall be the responsibility of the builder to install all of such devices, and to maintain them accurately in place until the concrete is placed.

*Released by the Standards Committee for convention consideration on a proposed motion for adoption as an ACI Standard.

Any and all disturbances of reinforcement from any cause whatsoever shall be fully corrected prior to the placing of concrete, and all damaged bar supports and spacers shall be repaired, or removed and replaced by the builder.

TABLE 1.—ONE-WAY SLAB CONSTRUCTION

Span	Rows of Slabs Bar Spacers for Panel	High Chairs and ⅝" Support Bars in Slabs 4" and Thicker*	
		Support Bars	High Chairs
0′ to 6′ Steel Continuous	1	1 Row at Beam	4′-0 o.c.
0′ to 14′ Steel Not Continuous	2	2 Rows at Beam	4′-0 o.c.
14′ to 20′ Steel Not Continuous	3	2 Rows at Beam	4′-0 o.c.
20′ to 26′ Steel Not Continuous	4	2 Rows at Beam	4′-0 o.c.

*Continuous high chairs may be substituted for individual high chairs and support bars.

Table 2.—ORDINARY BEAM AND JOIST CONSTRUCTION

(Bars 1 in. square and smaller)

Clear spans, beam (or joist)	Number of beam (joist) chairs					
	Single layer of bars	Two Layers		Three Layers		
		Lower	Top	Lower	Middle	Top
Over 0 ft. to 14 ft.....	2	2	2	2	2	2
Over 14 ft. to 23 ft.....	3	3	2	3	2	2
Over 23 ft. to 30 ft.....	4	4	3	4	3	2
Over 30 ft.....	(*)	(*)	(*)	(*)	(*)	(*)

*See Table 3.

TABLE 3.—HEAVY BEAM AND GIRDER CONSTRUCTION

(Beams or girders with large number of 1⅛- or 1¼-inch bars)

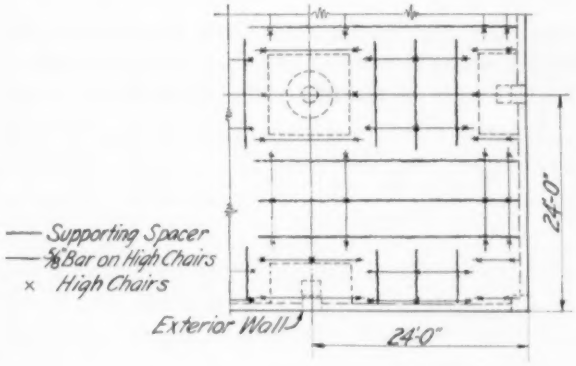
Clear Spans	Number of beam chairs					
	Single layer of Bars	Two Layers		Three Layers		
		Lower	Top	Lower	Middle	Top
Over 0 ft. to 14 ft.....	2	3	2	3	2	2
Over 14 ft. to 23 ft.....	3	5	2	6	2	2
Over 23 ft. to 30 ft.....	4	6	3	7	3	2
Over 30 ft. to 40 ft.....	6	7	4	8	4	3
Over 40 ft. to 50 ft.....	7	8	4	9	5	4
Spacing of beam chairs in spans other than above: All spans.....	7′0″	6′0″	10′0″	5′0″	8′0″	10′0″

TABLE 4.—FLAT SLABS

(Two and four way flat slabs—See Fig. 1 and 2)

Spans (center to center of Columns)	Supporting Spacers		High Chairs and 5/8" Support Bars under Ends of Bent Bars*			
	Column Strip or Direct Band	Middle Strip or Diagonal Band Bottom Layer	Support for Column Head Reinforcement		Support for Negative Reinforcement, Middle Strip	
			Support Bars	High Chairs	Support Bars	High Chairs
Over 0 ft. to 18 ft.	3	2	....	....	....	....
Over 18 ft. to 26 ft.	3	3	....	....	....	....
Over 26 ft. to 36 ft.	4	4	....	....	....	....
Around interior columns	....	....	2	8	....	....
Around exterior columns	....	....	2	5	....	....
Around corner columns	....	....	2	4	....	....
In interior panels	....	....	....	....	4	12
In exterior panels	....	....	....	....	5	15
In corner panels	....	....	....	....	6	18

*Continuous high chairs may be substituted for individual high chairs and support bars.



In roof slabs use one (1) more supporting spacer under column strips, or diagonal bands, and bottom layers of middle strips or diagonal bands, and one (1) more 5/8 in. support bar at interior column heads, than the number shown in Table 3.

Fig. 1—Two-way system

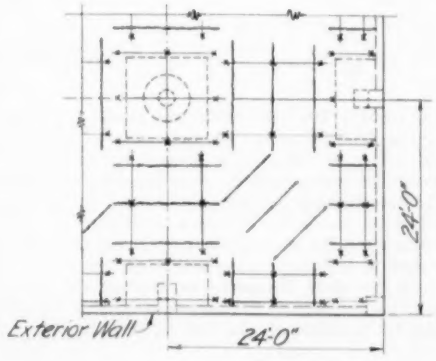


Fig. 2—Four-way system

TYPES OF DEVICES

Since various satisfactory types of supporting devices are on the market, this report makes no attempt to specify exact details and sizes. However, the following indicates the Committee's opinion as to a specification covering the most common types, which should produce devices meeting the requirements of paragraph 2, "Supports."

For supporting devices for slab steel in which the spanning member is a round section, either of cold drawn wire or of hot rolled steel having a yield point not less than 45,000 psi., the spacing of legs shall not exceed the following:

For continuous high chairs $s = 400\ d^3$

For continuous slab spacers, where the spanning member is not over $1\frac{1}{2}$ in. above the forms $s = 550\ d^3$

in which

s — spacing of legs in inches

d — diameter of round wire or rod in inches

The above rules shall apply to spacers having legs at a spacing not to exceed 30 in.

Supporting legs for continuous high chairs shall be so formed that the effective width of the support where it rests on the form shall not be less than $\frac{3}{4}$ of the height from the form to the top of the spanning member.

Supporting legs for continuous slab spacers shall be so formed that the effective width of the support, where it rests on the form, shall not be less than two times the height from the form to the top of the spanning member.

Legs of continuous high chairs made of wire or round rods shall be of such size that the ratio of height to radius of gyration shall not exceed 120 in the case of wires having a diameter less than $\frac{1}{4}$ in. or in case of all chairs which are supported upon metal forms.

In the case of continuous high chairs which are supported upon wood forms and in which the material of the legs is of a diameter of $\frac{1}{4}$ in. or larger the ratio of height to radius of gyration shall not exceed 156.

This report will be open for discussion at the 38th Annual Convention on a proposed motion by the Committee for the adoption of the report as an ACI Standard. Should the motion prevail by a majority vote of members present the report will be submitted by reference to letter ballot of ACI membership. "An amendment to the proposed Standard presented on the floor of the convention shall be sustained only when approved by nine-tenths of members voting." All discussion intended to influence convention action should therefore be available to the ACI Secretary in triplicate by Feb. 1, 1942 unless it is to have convention presentation by its author. Discussion is scheduled for publication in the JOURNAL for June 1942.

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Machine Methods for Trimming Subgrade and Placing Concrete Canal Lining*

By O. G. BODEN†

SYNOPSIS

The Bureau of Reclamation applied special machine methods to trimming subgrade and placing concrete lining on the Contra Costa Canal of the Central Valley Project. The subgrade trimming machine and the machine which placed, formed and vibrated the concrete, spanned the canal and rode on accurately placed rails. The results are satisfactory in maintaining accurate alignment and grade.

On the Contra Costa Canal of the Central Valley Project, which is under construction in California by the Bureau of Reclamation, machine methods used for trimming the subgrade and for placing concrete canal lining have led to precise alignment and grade. The cuts are excavated and the embankments made to the level of the berm at the top of lining with caterpillar tractors and carryall scrapers to within reasonably close limits. The fills are wet down to near optimum moisture by sprinkling and are compacted with a sheepsfoot roller or a pneumatic pavement breaker equipped with a tamping foot. The section is then leveled at berm elevation with a blade grader or carryall scraper to facilitate succeeding work.

Next, a survey party sets berm stakes offset three feet outside the line of the finished concrete lining on both sides of the canal. These berm stakes are set two-tenths of a foot below berm grade at 50-ft. intervals on the tangents and at 15-ft. centerline intervals around curves, to a tolerance of 0.005 of a foot. Rough excavation of the canal prism is made by dragline, and a template slightly smaller than the finished section is used as a guide by the dragline operator to permit working to close limits.

*Reviewed by the Institute Jan. 29, 1941.
†Construction Engineer, Bureau of Reclamation.

Fig. 1

A track consisting of a 50-lb. rail, supported by 12 x 20 x 2-in. ties at 4-ft. intervals, is laid on each side of the canal to an accurate line but with small variations in grade. The rails for the curves are bent on the job with a rail bender. Considerable care is taken in the alignment of the rails as they are used by both the subgrade trimming and the concrete canal lining machines.

The canal subgrade trimming machine consists of a triple steel truss which spans the canal and conforms to its size and shape (Fig. 1). It is supported at each end by two jacks resting on a two-wheeled truck whose wheels are double flanged and travel on the rails. A system of chain bucket excavators, elevators and belt conveyors trims the subgrade and delivers the trimmed material to the lower side of the canal. The trimming buckets are kept in line by guides designed to make the finished section conform to the theoretical subgrade surface.

Following subgrade trimming and placing of reinforcing steel, the concrete is placed, formed and vibrated in position by a double steel truss slipform canal lining machine which spans the canal and conforms to the size and shape of the finished section (Fig. 2). It is supported on the rails in a manner similar to the trimming machine and is fitted with jacks to provide adjustments in elevation. A weighted steel plate slip-

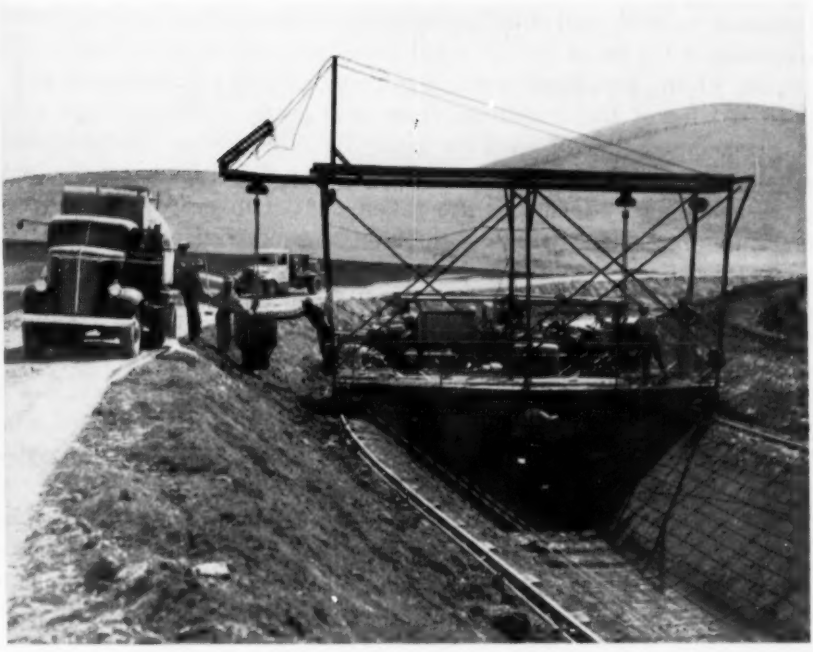

Fig. 2

form is towed behind the lining machine at a distance of from 15 to 50 ft. to iron out minor waves that occur in the concrete on the side slopes of the canal. This slipform leaves the concrete ready for troweling. In operation, the leading edge of the lining machine is carried ½ in. higher than the trailing edge to aid in consolidating the concrete. It has been found that any greater pitch has a tendency to float the 12-ton machine off the rails.

The proper grade for the trimming machine is obtained by using steel wire stretched on both sides of the canal between steel stakes and supported at intermediate points at a fixed height above grade. Control points on the four corners of the trimming machine are aligned with the wires, and adjustments to keep the machine on grade are made with the jacks previously mentioned. Frequent checking of the grade is necessary as the weight of the trimming machine (20 tons) is sufficient to slightly depress the track on soft ground. A spirit level fixed to the frame of the trimming machine aids in maintaining the machine level at all times. The elevation of the trimmed subgrade is checked with an engineer's level at frequent intervals along the canal as a further control. The trimming machine is kept on line by adequate staking of the track ties to prevent lateral movement. The weight of the trimming machine has

a tendency to settle and stabilize the track in place and so prevent further movement of the track by the canal lining machine which follows. The elevation of the stabilized track is obtained at 12½-ft. intervals and a record is kept of the deviations from true grade. The concrete canal lining machine is maintained on grade by reference to the previously recorded track deviations and necessary adjustments made by the jacks on the machine. Elevations are taken on the invert of the concrete canal lining immediately behind the lining machine at frequent intervals as a further check. Control of the alignment of the canal lining is maintained by the securely fastened rails and the only deviation likely to occur is a lateral movement, of approximately ⅜ in., of the flanged wheels on the rails.

Forward movement of both the trimming and lining machines is provided by two power winches mounted on each machine which wind up cables attached to deadmen set on both sides of the canal. Control of the movement around the curves is obtained by careful manipulation of these winches. Special care is taken to keep the wheel flange play of the lining machine in the same direction as that of the trimming machine so that the lateral deviation of the machines will be in the same direction.

The Contra Costa Canal is being constructed by contract and the Bureau of Reclamation forces set the lines and grades for the canal and inspect and check the completed work. The canal will be 46 miles long from the headworks at Rock Slough near Knightsen to a terminal reservoir near Martinez. It has an initial capacity of 350 second-feet and will convey fresh water from the Sacramento-San Joaquin delta to serve agriculture, industry, and a number of municipalities along the south shore of Suisun Bay.

Discussion of this paper should reach the ACI Secretary in triplicate by Mar. 1, 1942, for publication in the JOURNAL for June, 1942

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Job Problems and Practice

WHAT JPP MEANS

—an informal clearing house for mutual help on concrete problems; an opportunity for the ACI member who *knows* to tell the ACI member who *needs* the information. If the subject matter holds probabilities of general interest it will be briefed in these pages—with authorship credit unless the contributor prefers not.

If you know of an interesting problem whose solution has smoothed the way of someone in the field, record it briefly. If you have a problem, present it; a question, ask it. If you are moved to helpful comment or constructive criticism write the Secretary.

The "answers" are those of individuals to whom the questions are referred and not of the ACI.

A Comparison of Reinforced Concrete Tied Column Formulas (38-77)

By C. A. WILLSON*

Recommendations governing the design of reinforced concrete columns have undergone radical changes since 1937. This is particularly true of tied columns which now are permitted to carry much higher loads than formerly. In several instances these changes are sufficiently important to allow the use of tied columns where only spiral columns could be used under older specifications.

Certain increases in allowable safe load are due to changes in formula and others are due to specific provisions for the use of steels of high yield point.

The 1928 ACI Code has been chosen as representative of good practice of that period and several of the newer tied column recommendations are compared with it in the following tables and diagrams.

Graphs A and C are the same in Fig. 1 and 2 because $f_s = nf_c$. Since this steel stress is relatively low there would seem to be no objection to the use of a low yield point steel such as structural grade. This factor should be taken into account when making comparisons.

*Structural Engineer, State Architect's Office, Madison, Wisconsin

TABLE 1.—DESIGN FORMULAS AND LIMITS OF VERTICAL STEEL

Symbol	Code	Values of $\frac{P}{A}$	Limits of p_g
A	1928 ACI	$0.225f'_c\,[1 + (n-1)p_g]$	0.005 to 0.02
B	1937 Chicago	$0.20f'_c + 0.90\,f_s\,p_g$	0.01 to 0.04
C	1939 New York	$0.25f'_c\,(1 + np_g)$	0.005 to 0.04
D	1940 Joint Committee and 1941 ACI	$0.18f'_c + 0.80f_s\,p_g$	0.01 to 0.04

TABLE 2.—GRADES OF STEEL AND UNIT STRESSES USED IN FIG. 1

Symbol	Code	Grade of Steel	Unit Stress
A	1928 ACI	Not specified	nf_c
B	1937 Chicago	Intermediate grade and rail steel	15,000
C	1939 New York	Not specified	nf_c
D	1940 Joint Committee and 1941 ACI	Intermediate grade	16,000

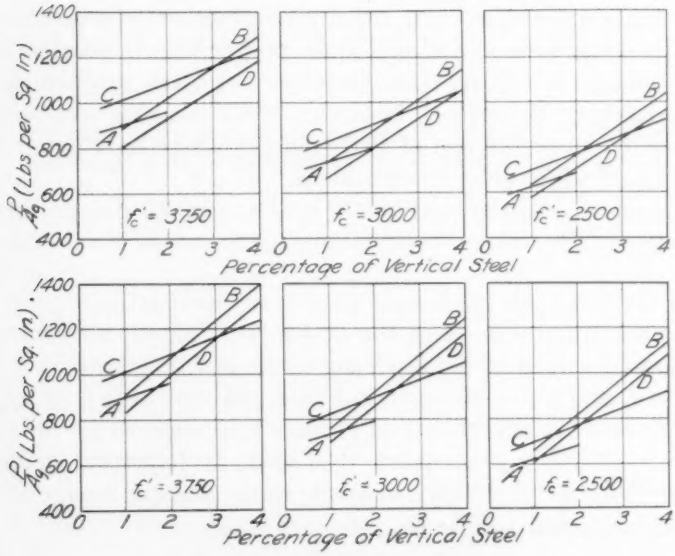

Fig. 1 (above) and 2 (below)

Graphs B in Fig. 1 representing intermediate grade and rail steel are nearly the same as Graphs D in Fig. 2 representing hard grade and rail steel. Many other interesting comparisons are possible.

TABLE 3.—GRADES OF STEEL AND UNIT STRESSES USED IN FIG. 2

Symbol	Code	Grade of Steel	Unit Stress
A	1928 ACI	Not specified	nf_c
B	1937 Chicago	Hard grade	18,000
C	1939 New York	Not specified	nf_c
D	1941 Joint Committee and 1941 ACI	Rail and Hard Grade	20,000

Aluminum Powder to Increase Bond of Concrete to Reinforcing Steel (38-78)

Q—Do small additions of aluminum powder increase bond in reinforced concrete?

BY F. R. McMILLAN*

A—We made a good many tests in our laboratory studying the influence of various factors on the bond between concrete and steel. In these studies we found that the bond resistance developed by horizontal reinforcing bars held rigidly in pull-out specimens in positions corresponding to those of the upper bars in deep beams decreased rapidly as the depth of the concrete under the bar increased. This decrease in bond has been shown to be mainly due to differences in the amount of settlement of the concrete under the bars.

We tried a good many ways to overcome this difficulty and one of the most effective methods was to add a very small amount of powdered aluminum to the concrete. The reaction of the aluminum with the cement releases a gas which produces a slight swelling in volume sufficient to overcome the normal settlement of the concrete under the fixed bars.

By this simple device we were able to develop practically the same bond resistance in the upper bars with as much as 33 in. of concrete under the bar as in corresponding lower bars with only two inches of concrete under the bar. Similar tests on beams have demonstrated the same low bond resistance of the upper bars without aluminum but the same marked improvement with aluminum additions as the pull-out test specimens.

*The question asked of and answered by F. R. McMillan, Director Research Laboratory Portland Cement Assn., Chicago, at the "Quiz Session" of the Institute's 37th Annual Convention, Washington, D. C., Feb. 18-20, 1941. See also companion question as to effect of aluminum powder on durability of concrete answered by Professor Carlson.

It should be pointed out in this connection that the use of aluminum powder increases the porosity of the cement paste and in plastic mixes decreases the compressive strength, this decrease being from 15 to 18 percent for quantities of powder normally required to offset settlement and to attain the improvement in bond resistance cited.

Effect of Aluminum Powder on Durability of Concrete (38-79)

Q—Is the durability of concrete impaired by small additions of aluminum powder?

BY ROY W. CARLSON*

A—The effect of small additions of aluminum powder on the durability of concrete is a subject that is now being investigated. According to the results of a few tests from two different laboratories, the aluminum powder did not decrease durability, but actually increased it slightly. In general, the tests paralleled those which Mr. McMillan mentioned† where bond to reinforcing steel was studied. In most cases, seven thousandths of one per cent of fine aluminum powder was added, by weight of cement. The durability tests consisted of alternate freezing and thawing of specimens of concrete until substantial length change or loss in strength developed. The improvement in durability due to the addition of aluminum powder was not large but there was some improvement.

Time Interval Between Successive Coats of Stucco (38-80)

Q—What time should elapse between the placing of successive coats of stucco?

BY W. G. KAISER‡

A—Specifications usually require that not less than 24 hours shall elapse between the application of the scratch (first) and the brown (second) coats provided the first coat has set firm and hard; otherwise a longer time is recommended. It is considered good practice to apply the finish coat not sooner than seven days after the preceding coat. An even longer period is desirable to make sure that the base coats have fully hardened and will not undergo further volume change. Any crazing which is likely to occur in the base coats should show up in seven days of hardening. Such crazing ordinarily can be effectively sealed with the finish coat.

*Associate Professor of Civil Engineering, Mass. Institute of Technology—one of the Board of Experts at the "Quiz Session" of the 1941 Convention.

†See elsewhere these pages another "Quiz Session" question and answer.

‡Manager Cement Products Bureau, Portland Cement Association, Chicago.

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Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Progress in cement research 1940

C. R. PLATZMANN, *Zement*, Vol. 30, Nos. 21, 22, 23, May 22, 29, June 5, 1941, pp. 273-275, 285-288, 297-301. Reviewed by L. T. BROWNMILLER

This article contains a review and a brief summary of the more important publications of the year on cement and cement manufacture.

Grand Coulee Dam nears completion

Engineering News-Record, Vol. 127, No. 11 (Sept. 11, 1941) pp. 91-92 Reviewed by S. J. CHAMBERLIN

By March 1942, 11,000,000 cu. yd. of concrete will have been placed. The five slots in higher portions of the dam, left open to take up "twist," are being filled with concrete in 50-ft. lifts to within 100 ft. of the reservoir surface. The 11-span concrete-arch highway bridge across the spillway is under construction.

Seawall and waterfront development completed at Corpus Christi

Engineering News-Record, Vol. 127, No. 11 (Sept. 11, 1941), pp. 72-74 Reviewed by S. J. CHAMBERLIN

The development includes about 12,000 ft. of reinforced concrete seawall. The stepped seawall is supported on timber piles and protected by steel sheet piling. Reinforcing for the supporting beams was assembled on a traveler and lowered into place. A covered traveler for the concreting operations provided protection against showers and sun.

Concrete core wall at Merriman Dam

Engineering News-Record, Vol. 127, No. 13 (Sept. 25, 1941) pp. 64-68 Reviewed by S. J. CHAMBERLIN

The cutoff wall consists mainly of twenty large concrete caissons sunk to rock with the nominal 18-in. spaces between caissons concreted solid after cleaning. Above the steel cutting edge, wood forms were used for the 14-ft. high working chamber, but above that steel forms were used inside and out. Concrete was distributed by pumping from a centrally located mixer plant.

Pneumatically placed concrete for walls of industrial plants

J. Q. A. GREENE, *Engineering News-Record*, Vol. 127, No. 13 (Sept. 25, 1941) pp. 73-74. Reviewed by S. J. CHAMBERLIN

Walls between columns may be inexpensively and quickly constructed of shotcrete. Steel members at openings should be designed to allow sash to be bolted or welded into place. Reinforcing steel consists of mesh attached to pencil rods stiffened with angles. The light plywood back forms (reusable) leave a satisfactory interior on the usual 2-in. walls.

First concrete paving 50 years old

TRACY BARTHOLOMEW, *Engineering News-Record*, Vol. 127, No. 9 (Aug. 28, 1941) pp. 57-59 Reviewed by S. J. CHAMBERLIN

After 50 years of service at Bellefontaine, Ohio, the pavement, which was laid in 5- and 6-ft. square blocks, shows its principal wear along the longitudinal joints where the steel tires of the horse-and-buggy days produced excessive abrasion. The transverse joints are sharp and in good condition. While placing the concrete the blocks were separated by tar paper and were not keyed. The production of the cement and the construction of the pavement are reviewed.

Sliding form used to construct walls of 77-ft. pit

Engineering News-Record, Vol. 127, No. 7, (Aug. 14, 1941) pp. 78-79 Reviewed by S. J. CHAMBERLIN

An inside sliding form 4 ft. high was used to line the pit excavated in shale rock. The pit was open on one side for the entire depth and above the 36-ft. level was open on two sides. Upward movement was accomplished by nine hand-operated screw jacks working on a 1-in. steel rod which was embedded in the concrete as work advanced. A platform above the form aided in setting steel, and two successive platforms below provided working room for the finishers. Concrete was dumped by bucket on the floor of the moving form and distributed by shovel.

Factors in aerial bombardment protection

HAROLD E. WESSMAN and WM. A. ROSE, *Engineering News-Record*, Vol. 127, No. 9, (Aug. 28, 1941) pp. 60-64. Reviewed by S. J. CHAMBERLIN

Designing buildings to resist bomb forces by the static-load method is unsatisfactory. With our present knowledge, designing for energy loads is equally unsatisfactory. Design should probably be based on observation and experimentation. Thick walls and floors to resist fragmentation, blast, and suction may provide shelter zones within the building. Pressure-time, pressure-distance and equivalent pressure-frequency curves from a British handbook are reproduced.

Structural problems in bomb protection

HAROLD E. WESSMAN and WM. A. ROSE, *Engineering News-Record*, Vol. 127, No. 11, (Sept. 11, 1941) pp. 75-78. Reviewed by S. J. CHAMBERLIN

In general, building construction should be designed (1) to minimize and localize the effects of direct hits, and (2) to resist the indirect effects of bombing. The latter governing in case of conflicting requirements. Wall-bearing construction offers little resistance to direct hits. Tables include recommended materials thicknesses against bomb fragments; fire-resistive ratings; thicknesses to resist blast and fragmentation; thicknesses for overhead protection; and debris loads. The design of bomb-resistance shelters is discussed.

Sixteen stories of concrete—six of steel

Engineering News-Record, Vol. 127, No. 13 (Sept. 25, 1941) pp. 57-60. Reviewed by S. J. CHAMBERLIN

The concrete design, based on the original steel design, was complicated by irregular column spacing. A successful design was obtained by using the rigid frame principle and controlled concrete with maximum column steel. Allowable live load reductions for upper stories was a factor. Irregular shaped cross-sections concealed in closets and partitions furnished the required area for columns. Loads from the setbacks were carried by a combination of steel and concrete construction. Above the 16th floor setback the original steel design was utilized. Two floors were constructed per 35-hr. week.

Moments in continuous rectangular slabs on rigid supports

L. C. Maugh and C. W. Pan, *Proc.* Am. Soc. C. E., Vol. 67, No. 5 (May 1941) p. 739-752.
Reviewed by H. J. Gilkey

Presents a method of analysis for rectangular slabs supported on four sides and continuous over the supports in which the entire panel is analyzed as a structural unit instead of using beam strips. Equations providing continuity between panels can be expressed in terms of the restraining moment at middle of each support. Use of sine distribution for edge moments simplifies solution and the results are more correct than those from use of the usual hypothetical beam strips. Equations can best be solved by successive approximations. Diagrams are provided for facilitating solutions. The method can be extended to cover various loading conditions by varying the constant terms in the equations.

A swimming pool tank built in a lake

Engineering News-Record, Vol. 127, No. 7 (Aug. 14, 1941) pp. 86-88 Reviewed by S. J. Chamberlin

In transforming the Aquacade of the New York World's Fair site into a modern swimming pool, two 60 x 65-ft. steel tanks were used as the outside forms for the reinforced-concrete bottom and side walls for the two diving areas. The bottom slab is 5½ to 7 ft. below the lake level and is supported by old and new piles. The tanks are of $\frac{5}{16}$-in. plate stiffened by rails. Skidded and held in exact position the tanks settled and came to rest as concrete was placed. Sloping inner-faces of the side walls required no forms. Above the level of the tanks side walls could be built in the dry by ordinary methods. For the shallow portion the wooden form for the bottom slab was supported on wooden piles cut off ½ ft. below water line. A wooden box was fitted around each pile and the concrete poured flush with top of boxes and allowed to set before the slab was constructed.

Crack prevention program—Hiwassee Dam

O. Laurgaard, *Proc.* Am. Soc. C. E., Vol. 67, No. 3 (March 1941) p. 327-349.
Reviewed by H. J. Gilkey

As the highest over-flow gravity dam (322 ft. from lowest foundation rock to roadway), this structure merited the special precautions adopted to prevent crack formation. The most distinctive construction feature was the coordinated program of precautions which attempted to apply to this one structure all of the means which have been discovered for limiting and controlling volume changes and the resultant cracking of the concrete. The special measures adopted were: (1) Use of low-heat cement; (2) low cement content; (3) thin casting lifts; (4) long exposure periods; (5) artificial cooling for mixing water; (6) washing, rinsing and cooling the aggregate; (7) artificial cooling of the concrete in place; (8) cleanup of horizontal joints between lifts; (9) use of steel reinforcement; (10) diagonal keyways on bulkhead joints; and (11) curing and winter protection. The interior of the dam was of leaner concrete than were the faces but transition was made gradual by planned overlapping. Special measures were adopted in the vicinity of projecting foundation rock to secure bond and to avoid cracking at such junctions. Maximum temperatures were lowered an estimated 31 deg.; cracking was almost eliminated, and the saving in cement more than offset the added expense of the measures employed.

Forming details and practices for architectural concrete

A. J. Boase, *Engineering News-Record*, Vol. 127, No. 7 (Aug. 14, 1941) pp. 92-94
Reviewed by S. J. Chamberlin

The condition of the forms at the time the cement sets determines the appearance of the completed job. Better workmanship and somewhat better materials are required

than for ordinary work. Tongued and grooved lumber, shiplap for sheathing and 4 x 4 studs and horizontal 2 x 4's at plywood joints are needed to prevent leakage. 2 x 6 wales projecting beyond corners with vertical kick strips for wedging and having the sheathing in one direction run through the corner help to secure tight corner joints. At openings the ties should be inside the opening in order to draw the sheathing tight against the bucks. Studs should not be on more than 16-in. centers for 1-in. sheathing or $\frac{11}{16}$-in. plywood and not more than 12-in. centers for $\frac{5}{8}$-in. plywood. Wales should be on not more than 24-in. centers with vertical wales placed outside the horizontal ones at about 8-ft. intervals. Allowance for swelling at openings by cutting the bucks $\frac{3}{4}$ in. short and then tacking a $\frac{3}{4}$ x 2-in. strip across the end facilitates form removal. Construction joints should be provided at top of door and window openings so that the concrete can be placed to this level and left to harden. The construction joint must be a straight, level line. Curved wall forms require minimum stud spacing or horizontal ribs.

Extensometer measurements, North Avenue bridge, Chicago, Ill.

LAWRENCE T. SMITH and PAUL LILLARD, *Proc.* Am. Soc. C. E., Vol. 67, No. 5 (May 1941) p. 781-803.
Reviewed by H. J. GILKEY

The design of this important skew two-hinged rigid frame reinforced concrete bridge carrying heavy Outer-Drive traffic over the also heavy LaSalle St. traffic in Lincoln Park, Chicago, involved a number of assumptions on which checks were desired. From tests of auxiliary compressive specimens the modular ratio between steel and concrete was taken as ten. Strains were measured at 28 strain gage stations built into the deck girders, tie slab and abutment legs of the center-line interior frame and the east fascia frame. A specially constructed 20-in. strain gage was used on plugs set in the concrete and on partially exposed bars.

Loading studies included dead load conditions and four cases of live loading. Temperature effects were studied for about 4 months, the range of concrete temperatures being 78 F. to 16 F. for air temperatures of 96 F. to -15 F. The response to changes in air temperature was relatively quick.

Following the removal of deck centering after 81 days, there was a 6-day period of structural readjustment. After structural equilibrium was reached there were movements from temperature changes of the air. There was very little effect from the passive resistance of backfill or restraint of subpiers. A cut-bar technique was introduced to check strain measurement under dead load. After the strain observation was made, the bar was isolated from concrete adherence and cut outside the gage length, releasing the strain completely. The bar was later restored by welding at the cut.

From the various live load tests it was concluded the structure functions as an elastic two-hinged tied frame and the observed stresses accord well with the indications of elastic theory. Appended to the paper is a list of 46 references, largely on strain gages and their use on structures.

Influence of steel grade upon strength and deflections of reinforced concrete beams

E. SUENSON—53 pages, *Ingeniorvidenskabelige Skrifter* 1940 No. 3 Danish Engineers Society.
Reviewed by BENGT FRIBERG

The tests included six $8\frac{1}{2}$ x $8\frac{1}{2}$ in. concrete beams for 14-ft. 10-in. span reinforced with 2 bottom bars and $\frac{1}{4}$-in. stirrups spaced 8-in. c to c. Three grades of steel for two beams each had yield points of 39,000 psi., 50,000 psi., and 55,000 psi., (the last being Isteg steel with yield point determined as stress for .4 per cent total elongation). The Modulus of Elasticity was for the three types of reinforcement 30.0, 29.5 and 20.5 million

psi. The reinforcing percentages were for the three steels 1.233, .978 and .933 per cent, respectively.

Based on triangular stress distribution in the compression zone, failures corresponded to 114, 111 and 117 per cent, respectively, of the steel yield point. Based on rectangular stress distribution and compressive stress equal to the concrete cube strength, computed steel stresses at failure were: 102, 102 and 108, respectively, of the steel yield point (the slight variation for Isteg Steel is the result of the arbitrary establishment of the yield point stress for that steel).

Careful measurements of crack widths and crack lengths were made. The first cracks, observed by means of a microscope, appeared at a fiber elongation of .01 per cent with almost no variations for the different reinforcing grades. Under magnifying glass, the cracks were visible first at about 30 per cent higher load. The crack development was not audible, but later crunching seemed associated with loss of bond between the concrete and steel near the cracks. The neutral axis is at or somewhat below the center of the beam up to the first crack, and thereafter remains somewhat above the position computed for triangular stress diagram, until near the ultimate when it moves considerably further toward the beam compression side.

Deflection curves were carefully observed for the $\frac{1}{3}$ point test loads and for a given beam may well be replaced by two straight lines corresponding to the cracked and uncracked beam sections. The conventional deflection increment formula $K\frac{P\ L^3}{E\ I}$ could be used for both stages if the factor $E \times I$ was manipulated using for uncracked sections $n = 7.5$; and for the cracked section $n = 15$ in computing Moments of Inertia and for the modulus of elasticity E for the cracked section a value of .5 of the E value for uncracked concrete, if rolled reinforcement is used, and .4 of the original value, if Isteg Steel is used.

At a desired uniform factor of safety, the formulas for permissible loads are dependent not only upon the steel yield point but upon the concrete strength as well, when steels with varying strengths are used, because of the possible increase in moment arm with increasing concrete strength at loads near steel failures.

The conclusion was that the sole advantage of high yield point steel lay in the weight saving of steel, requiring in each instance consideration of the proportion in price between weaker and stronger steels to reach maximum economy. As the crack width grows with the steel stress, the high yield point steels at higher working stress should be used primarily indoors where rusting is precluded.

Buildings damaged by air raids and notes relative to reconstruction

Report of Science Committee, Oscar Faber, Chairman, Institution of Structural Engineers, London. (Copyright—April 1941; liberally abstracted by special permission of I. S. E.)

A. C. I. Editorial Abstract

From the foreword: "In the preparation of this Report the Committee has had before it the problem of presenting as much information as it can without giving away anything which would be likely to be useful to the enemy. For this reason it has been necessary to suppress any reference to the names or positions of individual buildings, the size and nature of the bomb which caused the destruction and any other references from which information of this character might be gleaned."

At the request of the Council of the Institution of Structural Engineers, the Science Committee prepared a report which embodies the experience collected by some of its members, and others, in dealing with the reconstruction and repair of buildings damaged by air raids.

Damage is so varied that it is rare to find any similarity between one case and another, and it is extremely difficult to lay down any general rules for the guidance of those who are asked to give technical assistance in the work of repair and reconstruction.

Examination by a qualified engineer should be one of the first steps to be taken when a structure has been damaged, and this examination should be made previous to any demolition or repair. This course will frequently prevent further damage through demolition or repairs without an instructed knowledge of the forces to be dealt with.

Impressed by the variety of problems the Committee has concluded that a report dealing with every aspect of the matter which an engineer would be called upon to consider in devising schemes for reconstruction and repair could not fall much short of a complete compendium of structural engineering in all its aspects. The following cases can be cited:

(*a*) A steel column can be badly bent or otherwise damaged and a detailed knowledge of the strength of steel columns to carry direct loads combined with eccentric loadings will obviously be required. (*b*) A reinforced concrete beam may be badly cracked or broken, and clearly a detailed knowledge of reinforced concrete design and construction will be needed before useful and dependable advice can be given.

(*c*) Foundations may be destroyed or badly undermined as the result of a bomb penetrating the ground for a considerable distance, disturbing the sub-soil under the foundations, both in the actual crater produced by the explosion and by the shock from the explosion, which is transmitted to the surrounding foundations, loosening the soil on which other foundations depend.

This disturbance, being less obvious than the damage to the superstructure, is more likely to be overlooked. Much of the damage, attributed to blast is really due to overturning, by upheaval of the ground below the foundations. Moreover, the most conclusive evidence is often of a kind which is likely to be unintentionally destroyed, and this is another reason for calling in a qualified engineer before beginning demolition, clearing of the site, or repairs.

Although every formation has its own peculiarities in this matter of ground disturbance the London clay and other clays point very clearly to its wide extent. The curious, almost cubical fracture displayed by the large lumps to be seen in the craters, suggest that this kind of fissure is by no means confined to the lumps that are actually ejected, but that the clay generally, over a wide area, is probably intersected by similar fissures with a varying degree of relative movement of the separated "blocks "of clay. One can imagine that near the point of explosion the condition is similar to that of a badly shaken brick wall, while further from it there is merely cleavage, without much movement.

(*d*) Burst water mains may cause flooding and erosion of foundations and the soil may be loosened and rendered soft and unsuitable for foundations for a considerable distance from the original disturbance.

(*e*) Excavations may be necessary for effecting the repairs to the services such as water mains, gas mains, electric mains, etc., and in excavating through the soil which has become unsuitable for foundations, difficult problems frequently arise in carrying out such work so as not to cause further disturbance to existing foundations in the immediate neighbourhood below the level of which the new excavations have to be taken.

All these problems obviously involve a thorough knowledge of soil mechanics, the safe bearing pressures on different soils, the pressures on timbering, the precautions neces-

sary when timbering to prevent the soil, on which foundations may be resting, from yielding, and many other problems of a similar character.

It is beyond the scope of this report to prepare a compendium of structural engineering which would be wide enough to deal with all these problems.

As far as the structural aspect of reparations is concerned the primary requirement is that structural engineers with an all-round experience and wide knowledge of their subject should be employed.

One important and difficult problem relates to reinforced concrete beams, in which the sudden shock of explosion has burst away the concrete cover from the reinforcements. In some cases a minute space shows between the main rods and the surrounding concrete, clearly indicating that the bond resistance or adhesion has been badly damaged. How much strength can still be credited to such a structure and what form of repair is to be undertaken become highly technical questions.

In many cases the engineer must decide whether a damaged member may be removed for repair or replacement. Skilful use of strutting and timbering may become necessary to avoid the transfer of loads to other members which were not designed to receive them.

The problem often involves a recalculation of the strength of the structure on the basis of its actual loads, which may differ considerably from its original design loads, and demands the acceptance as a temporary measure of stress values considerably higher than ordinary working stresses.

Structures with a liberal factor of safety in their original design stand up much better than structures in which the original margin was reduced to a minimum, and are, as a rule, easier to deal with as they permit members to carry loads in excess of those for which they were originally designed, in a manner which is impossible where little or no margin existed.

In the Institution of Structural Engineers' A.R.P. Report, published in October, 1938, great stress was laid on the undoubted superiority against aerial attack of steel frame buildings and reinforced concrete frame buildings with concrete floors in comparison with the old-fashioned type of building. Experience has more than justified those recommendations.

The behaviour of framed buildings, both steel and concrete, has proved to be very satisfactory and it is the opinion of the Committee that such structures have stood up to attack in the most remarkable manner and have shown great powers of resistance and resilience. They have withstood attacks from bombs of the heaviest calibre where buildings with brick walls and timber floors have collapsed utterly under the action of small bombs.

A typical picture of an old-fashioned building after a direct hit from a small bomb is a mass of bricks and crushed timber forming a heap on the ground, whereas a typical picture of a steel frame building or a building of reinforced concrete subject to a direct hit is a building still standing, though with wall panels and partitions blown out and some of the main structural elements broken, bent or otherwise seriously damaged, but nevertheless the structure still standing and still giving protection to persons in the lower stories.

From the experience of air raid damage up-to-date, it is clear that other things being equal, floors of solid concrete strengthened with filler joists or steel reinforcement stand up better and give more protection against bombing than floors in which lightness has been obtained by the use of hollow tiles or otherwise.

Evidence shows that massive and heavy construction in walls, floors and roofs increases the resistance to blast, and protection given against penetration by shrapnel, splinters or falling debris.

For buildings of the future the Committee feels that framed buildings of steel or of reinforced concrete with reinforced concrete floors deserve to be adopted, even when no advantage in cost is apparent, owing to their resilience and resistance to all kinds of forces for which they were not primarily designed, but to which they may in the future become subject.

The value of soundly jointed structures has become clear.

Other things being equal, structures in which all the beams are properly connected to the columns in accordance with good modern practice are to be preferred to structures in which members are free to act as independent units.

In the latter case, if a member is severed by a direct hit or otherwise, it may fall and take with it any construction which it supports, such as floor slabs, partitions and the like.

In the case of a connected or continuous structure, a member may be completely severed and yet, apart from the portion so severed, the ends may still continue approximately in their original positions and carry load by cantilever action from their supports. This is particularly true in the case of the main beams in single storey factory construction and may avert the dire results of what may be termed "spreading collapse"; that is, parts of a structure adjacent to the damaged or broken portion being dragged down by the collapse of the latter, the effect spreading over a considerable area. In the secondary members and truss connections of such structures, where continuity or rigidity is often impracticable, it has been shown that frequent systems of bracing are invaluable in localizing the effects of broken or collapsed members, and while parts of the surrounding structure left standing may thus be temporarily subjected to stresses greatly in excess of those normally adopted in design, the damage can quickly be repaired and the structure brought into a serviceable condition. This is largely due to the fact that, at the time of the failure of the primary member, the adjacent members may not be subjected to the full load for which they were designed, as no doubt allowance would have been made for wind, snow, crane loads, etc., all taken at their maximum value but which may be non- existent or partially existent at the time of the accident.

It is frequently found that mild steel members such as beams and stanchions are often bent as the result of bombing and of subsequent fires. In the case of mild steel complying with B.S.S. 15, such steel members may normally be used again after straightening provided they have not been obviously burnt or oxidized to a degree whereby the original section of metal is seriously impaired.

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Proposed Recommended Practice for the Design of Concrete Mixes*

REPORTED BY ACI COMMITTEE 613

R. F. BLANKS
Chairman

E. N. VIDAL
Secretary

ROY W. CARLSON
HERBERT COFFMAN
W. M. DUNAGAN†
FRED HUBBARD
F. H. JACKSON
W. F. KELLERMANN
H. L. KENNEDY
GUY H. LARSON
T. C. POWERS
E. B. RAYBURN
H. C. ROSS
I. L. TYLER
STANTON WALKER

INTRODUCTION

The design of concrete mixes is the determination of the most economical and practicable combination of available aggregates, cement, water, and, in some cases, admixture that will produce a mixture having the required degree of workability and which will develop required qualities on hardening.

The most practical procedure for determining the final proportions of concrete for a given purpose is actual trial and adjustment on the job. The problem thus becomes a matter of selecting a trial mix for starting concrete operations that will require the least adjustment on the job.

The design of trial concrete mixes can be most effectively accomplished by laboratory investigation of the concrete-making properties of the particular materials to be used. For important work, for unusual conditions or materials, or where flexural strength of the hardened concrete is a controlling factor, such laboratory tests are especially desirable.

*The report, in its present form was received by the Institute Dec. 8, 1941 after final ballot of Committee 613 and release by Standards Committee as information and for discussion. The result of the most recent ballot in the originating committee, after successive revisions, was: 13 members approved; one disapproved; one member did not vote.

†Professor Dunagan died Nov. 24, 1941.

When it is impracticable to conduct laboratory concrete tests, trial mixes for starting concrete operations can be selected by the judicious application of known basic concepts and established empirical relationships.

RECOMMENDED PROCEDURE FOR THE DESIGN OF CONCRETE MIXES

1. Select the water-cement ratio from test data, experience, or established relationships, to meet the specified requirements for durability and strength (compression or flexure).

2. Select the limits of slump that will permit proper handling and consolidation of the concrete under the job conditions involved.

3. Determine the largest maximum size of available aggregate suitable for use under the job conditions involved.

4. Estimate from test data, experience, or established relationships, the minimum percentage of sand that will provide the proper degree of workability.

5. Estimate the amount of water per cubic yard of concrete that will be required to fulfill the conditions of 2, 3, and 4.

6. Compute the trial-mix proportions that will satisfy the conditions established in steps 1 to 5. Make such adjustments in the trial mix on the job as may be necessary.

LABORATORY TESTS

Wherever practicable, the various relationships needed in the procedure for designing concrete mixes should be established by laboratory tests of concrete made with the materials to be used in the work. The extent of the laboratory investigations for any given job will depend on the size, importance, and service conditions of the structures involved. Therefore, no attempt is made in this report to discuss the details of laboratory test programs. The minimum laboratory determinations that will permit effective trial-mix design are tests of aggregates involving determinations of grading, specific gravity, absorption, and total moisture content.

Important factors which should be determined by tests of concrete mixes in the laboratory include:

1. Relation between water-cement ratio and strength (compression or flexure),

2. Variations in consistency (slump) and workability characteristics for various combinations of the ingredient materials, and

3. Unit water content and cement requirement for various consistencies and aggregate gradings.

TABLE 1—NET WATER-CEMENT RATIOS FOR VARIOUS TYPES OF CONSTRUCTION AND EXPOSURE CONDITIONS**

Type or Location of Structure	Severe or Moderate Climate, Wide Range of Temperature, Rain, and Long Freezing Spells or Frequent Freezing and Thawing										Mild Climate, Rain or Semiarid Rarely Snow or Frost									
	Thin Sections				Moderate Sections				Heavy and Mass Sections		Thin Sections				Moderate Sections				Heavy and Mass Sections	
	Reinf.		Plain		Reinf.		Plain				Reinf.		Plain		Reinf.		Plain			
	By Weight	Gal./Sack	By Weight	Gal./Sack	By Weight	Gal./Sack	By Weight	Gal./Sack	By Weight	Gal./Sack	By Weight	Gal./Sack	By Weight	Gal./Sack	By Weight	Gal./Sack	By Weight	Gal./Sack	By Weight	Gal./Sack
A. At the water line in hydraulic or water-front structures or portions of such structures where complete saturation or intermittent saturation is possible, but not where the structure is continuously submerged:																				
In sea water	0.44	5	0.49	5½	0.49	5½	0.53	6	0.53	6	0.44	5	0.49	5½	0.49	5½	0.53	6	0.53	6
In fresh water	0.49	5½	0.53	6	0.53	6	0.58	6½	0.58	6½	0.49	5½	0.53	6	0.53	6	0.58	6½	0.58	6½
B. Portions of hydraulic or water-front structures some distance from the water line, but subject to frequent wetting:																				
By sea water	0.49	5½	0.53	6	0.53	6	0.53	6	0.53	6	0.49	5½	0.58	6½	0.58	6½	0.62	7	0.62	7
By fresh water	0.53	6	0.58	6½	0.58	6½	0.58	6½	0.58	6½	0.53	6	0.62	7	0.62	7	0.67	7½	0.67	7½
C. Ordinary exposed structures, buildings and portions of bridges not coming under above groups	0.53	6	0.58	6½	0.58	6½	0.62	7	0.62	7	0.53	6	0.62	7	0.62	7	0.67	7½	0.67	7½
D. Complete continuous submergence:																				
In sea water	0.53	6	0.58	6½	0.58	6½	0.62	7	0.62	7	0.53	6	0.58	6½	0.58	6½	0.62	7	0.62	7
In fresh water	0.58	6½	0.62	7	0.62	7	0.67	7½	0.67	7½	0.58	6½	0.62	7	0.62	7	0.67	7½	0.67	7½
E. Concrete deposited through water	*	*	*	*	0.49	5½	0.49	5½	0.49	5½	*	*	*	*	0.49	5½	0.49	5½	0.49	5½
F. Pavement slabs directly on ground:																				
Wearing slabs	0.49	5½	0.53	6	*	*	*	*	*	*	0.53	6	0.58	6½	*	*	*	*	*	*
Base slabs	0.58	6	0.62	7	*	*	*	*	*	*	0.62	7	0.67	7½	*	*	*	*	*	*

G. Special case: (a) For concrete not exposed to the weather, such as interiors of buildings and portions of structures entirely below ground, no exposure hazard is involved, and the water-cement ratio should be selected on the basis of the strength and workability requirements.

*These sections not practicable for the purpose indicated.
**Adapted from Table 1 of the 1940 Joint Committee Report on Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete.

In addition to supplying data for a more exact design of trial mixes, the laboratory tests provide useful information for adjusting concrete mixes in the field and for determining the relative costs of concrete made with different combinations of materials.

SELECTION OF WATER-CEMENT RATIO

The water-cement ratio should be selected on the basis of (a) the required durability, and (b) the strength specified.

The maximum allowable water-cement ratio for the type of construction and exposure condition involved should be selected from Table 1. The water-cement ratio for the specified strength should be determined by laboratory tests. When it is not practicable to make such tests, the water-cement ratio required for the specified *compressive* strength may be taken directly from Table 2, which gives a series of conservative values derived from a large number of tests with typical materials.

TABLE 2—COMPRESSIVE STRENGTH FOR VARIOUS WATER-CEMENT RATIOS*

Net Water-Cement Ratio		Probable Strength at 28 Days (Pounds per Square Inch)
By Weight	Gallons per Sack of Cement	
0.44	5	5,000
0.49	5½	4,500
0.53	6	4,000
0.58	6½	3,600
0.62	7	3,200
0.67	7½	2,800
0.71	8	2,500
0.75	8½	2,000

*Adapted from Table 2 of the 1940 Joint Committee Report on Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete.

A corresponding table of values for flexural strength is not given because of the wider range in flexural strength which may be obtained with a given water-cement ratio for different materials. Where flexural strength is specified, the required water-cement ratio should be determined by laboratory tests, as previously stated.

The lower of the two water-cement ratios, (a) that required for durability as shown in Table 1, or (b) that required for the specified strength as determined by test or from Table 2, should always be used.

SELECTION OF LIMITING SLUMP AND MAXIMUM SIZE OF AGGREGATE

It is advisable to use the lowest slump compatible with proper placing. Table 3 gives the limiting values recommended for various types of construction.

TABLE 3—RECOMMENDED SLUMPS FOR VARIOUS TYPES OF CONSTRUCTION*

Type of Construction	Slump in Inches**	
	Maximum	Minimum
Reinforced foundation walls and footings	5	2
Plain footings, caissons, and substructure walls	4	1
Slabs, beams, and reinforced walls	6	3
Building columns	6	3
Pavements	3	2
Heavy mass construction	3	1

*Adapted from Table 4 of the 1940 Joint Committee Report on Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete.
**When high-frequency vibrators are used, the values given should be reduced about one-third.

For economic and other reasons, the maximum size of aggregate should be as large as practicable and available but should not exceed two-thirds of the minimum clear distance between reinforcement. Recommended limits are given in Table 4.

TABLE 4—MAXIMUM SIZE OF AGGREGATE RECOMMENDED FOR VARIOUS TYPES OF CONSTRUCTION

Minimum Dimension of Section (Inches)	Maximum Size of Aggregate*, in Inches, for:			
	Reinforced Walls, Beams, and Columns	Unreinforced Walls	Heavily Reinforced Slabs	Lightly Reinforced or Unreinforced Slabs
2½ - 5	½ - ¾	¾	¾ - 1	¾ - 1½
6 - 11	¾ - 1½	1½	1½	1½ - 3
12 - 29	1½ - 3	3	1½ - 3	3
30 or more	1½ - 3	6	1½ - 3	3 - 6

*Based on square screen openings.

ESTIMATING THE PERCENTAGE OF SAND AND UNIT WATER REQUIREMENT

For a given set of materials and water-cement ratio, the unit water content (water required for a given volume of concrete) is the next most important basic factor affecting the quality of concrete. The optimum percentage of fine aggregate for a concrete mix is that quantity which will result in the lowest unit water content and also provide the required degree of workability, with an adequate margin of safety to prevent difficulties from variations in working conditions and materials. Obviously, for a given water-cement ratio, the mix requiring the lowest water content will also require the least amount of cement.

The optimum percentage of fine aggregate can be determined to the best advantage on the job under actual working conditions. However, laboratory concrete tests with the materials to be used in the work

TABLE 5—APPROXIMATE PERCENTAGE OF FINE AGGREGATE AND APPROXIMATE TOTAL WATER CONTENT PER CUBIC YARD OF CONCRETE

Grading of Fine Aggregate (Minus No. 4 Sieve)	Maximum Size of Coarse Aggregate	Rounded Coarse Aggregate			Angular Coarse Aggregate		
		Fine Aggregate Per Cent Total Aggregate*	Net Water Content		Fine Aggregate Per Cent Total Aggregate*	Net Water Content	
			Pounds per Cubic Yard	Gallons per Cubic Yard		Pounds per Cubic Yard	Gallons per Cubic Yard
Fine	¾ inch	41	310	37	46	335	40
	1½ inch	34	280	34	39	305	37
	3 inch	28	250	30	33	275	33
(F.M. 2.20—2.60)	6 inch	23	220	26	28	245	29
Medium	¾ inch	44	310	37	49	335	40
	1½ inch	37	280	34	42	305	37
	3 inch	31	250	30	36	275	33
(F.M. 2.60—2.90)	6 inch	26	220	26	31	245	29
Coarse	¾ inch	47	310	37	52	335	40
	1½ inch	40	280	34	45	305	37
	3 inch	34	250	30	39	275	33
(F.M. 2.90—3.20)	6 inch	29	220	26	34	245	29

The above values are based on the use of reasonably well graded materials having average characteristics and also on average mixes having a slump of about three inches.

For stone sand, increase the tabulated percentages of sand by about 3 per cent and the water content by about 15 pounds.

Increase or decrease the water content three per cent for each increase or decrease of one inch in slump.

*By absolute volume. When the fine and coarse aggregate have about the same specific gravity, the percentages also apply by weight.

will provide information permitting a close approximation of the proper proportion. The Committee recognizes that there are many excellent methods for proportioning fine and coarse aggregate for trial concrete mixes. Where conditions or personal preference dictates, use may be made of any of the basically sound methods described in the current literature.

It is recommended that the proportions for trial concrete mixes be determined on the basis of the estimated unit water requirements. Where laboratory tests cannot be made, table 5 may be employed. The values given in Table 5 are average values for aggregate gradings within acceptable limits and were derived from a composite of data, information, and experience from many sources.

COMPUTATION AND ADJUSTMENT OF TRIAL MIX

Assume that a trial concrete mix for starting field operations is required for a heavily reinforced retaining wall having a minimum thickness of eight inches. The wall will be subjected to severe climatic exposure and has been designed on the basis of a compressive strength at 28 days of 3,500 psi. The concrete is to be placed by vibration.

Whenever possible, the water-cement ratio which will fulfill the requirement for strength should be determined by laboratory tests on concrete made with the materials to be used in the work. However, for the purpose of this example it will be assumed that such tests are not practical and that an average value must, therefore, be used. It will also be assumed that no trial batches are to be made in the laboratory and that the following information regarding the materials to be used in the work is available:

Cement—Average brand, standard type.
Specific gravity—3.15 (assumed).
Fine aggregate—River sand, wet.
Medium fineness and grading.
Specific gravity—2.65 (saturated, surface dry).
Free moisture content—5 per cent by weight.
Coarse aggregate—Crushed stone, air-dry.
Reasonably well graded, angular.
Specific gravity—2.55 (saturated, surface dry).
Absorption of water—Held to negligible value by sprinkling before batching.

From Table 1 it is found that a water-cement ratio of 0.53 is the maximum which should be used for the type of structure and service conditions involved. From Table 2 it is found that a water-cement ratio of about 0.58 by weight will be required to provide the design strength of 3,500 psi. The value of 0.53, being the lower, is used. From Tables

3 and 4 it is concluded that a three-inch slump will be satisfactory and that 1½-in. maximum aggregate can be used.

From the information given in Table 5 it is estimated that the percentage of sand for the trial mix should be 42 per cent, by absolute volume of the total aggregate, and that 305 lb. (36.5 gal.) of water will be required per cubic yard of concrete. (Actually the water content may be more or less than the estimated amount as determined by the amount of water required for a three-inch slump when a trial batch is mixed on the job.)

$$\text{Cement content} = \frac{\text{Net water content}}{\text{water-cement ratio}}$$

$$= \frac{305}{0.53} = 575 \text{ lb. per cu. yd.}$$

$$= \frac{575}{94} = 6.12 \text{ sacks per cu. yd.}$$

$$\left(\begin{matrix}\text{Absolute volume,}\\ \text{water + cement}\end{matrix}\right) = \frac{\text{Water content}}{62.4} + \frac{\text{Cement content}}{\text{specific gravity} \times 62.4} =$$

$$\frac{305}{62.4} + \frac{575}{3.15 \times 62.4} = 7.81 \text{ cu. ft. per cu. yd. of concrete.}$$

$$\left(\begin{matrix}\text{Absolute volume,}\\ \text{total aggregate}\end{matrix}\right) = 27 - \left(\begin{matrix}\text{absolute volume,}\\ \text{water + cement}\end{matrix}\right)^{*}$$

$$= 27 - 7.81 = 19.19 \text{ cu. ft. per cu. yd. of concrete.}$$

$$\left(\begin{matrix}\text{Absolute volume,}\\ \text{fine aggregate}\end{matrix}\right) = \text{percent fine aggregate} \times \left(\begin{matrix}\text{absolute volume,}\\ \text{total aggregate}\end{matrix}\right)$$

$$= 0.42 \times 19.19 = 8.06 \text{ cu. ft. per cu. yd. of concrete.}$$

$$\left(\begin{matrix}\text{Absolute volume,}\\ \text{coarse aggregate}\end{matrix}\right) = \left(\begin{matrix}\text{absolute volume,}\\ \text{total aggregate}\end{matrix}\right) - \left(\begin{matrix}\text{absolute volume,}\\ \text{fine aggregate}\end{matrix}\right)$$

$$= 19.19 - 8.06 = 11.13 \text{ cu. ft. per cubic yd. of concrete.}$$

$$\left(\begin{matrix}\text{Fine aggregate}\\ \text{content}\end{matrix}\right) = \text{absolute volume} \times \text{specific gravity} \times 62.4$$

$$= 8.06 \times 2.65 \times 62.4 = 1{,}333 \text{ lb. per cu. yd. of concrete.}$$

$$\left(\begin{matrix}\text{Coarse aggregate}\\ \text{content}\end{matrix}\right) = 11.13 \times 2.55 \times 62.4 = 1{,}771 \text{ lb. per cu. yd. of concrete.}$$

$$\text{Trial-mix proportions} = \frac{575}{575} : \frac{1{,}333}{575} : \frac{1{,}771}{575}$$

$$= 1 : 2.3 : 3.1$$

The equipment available for performing the work includes a three-sack batch mixer equipped with a water tank calibrated in gallons and also a water meter graduated in pounds, and batching scales for weighing the separated sizes of fine and coarse aggregate.

Cement is to be batched on the basis of integral sacks and the batch will contain three sacks, or $3 \times 94 = 282$ lb. of cement, which is a fixed quantity. The batch weights of the fine and coarse aggregate for the trial mix will be:

$$\text{Fine aggregate} = 1{,}333 \times \frac{282}{575} = 654 \text{ lb. (net), to which must be added the weight}$$

of the free moisture in the sand = 654 + (0.05 × 654) = 687 lb. total.

*Air voids assumed to be negligible. When the air content is appreciable, as when air entraining agents are used, a suitable allowance should be made considering the air as an additional ingredient.

$$\text{Coarse aggregate} = 1{,}771 \times \frac{282}{575} = 869 \text{ lb.}$$

A few batches of concrete are mixed on the job, and the average quantity of water per batch for the desired three-inch slump is found to be 13.1 gallons or $13.1 \times 8.34 = 109$ lb.

The free water contained in the sand is:

$$0.05 \times 654 = 33 \text{ lb.}$$

The net water in the mix is:

$$109 + 33 = 142 \text{ lb.}$$

The net water-cement ratio is:

$$\frac{142}{282} = 0.50 \text{ by weight.}$$

Since a water-cement ratio of 0.53 is desired (water content of $0.53 \times 282 = 150$ lb. for a three-sack batch), the mix requires less water than was estimated.

The absolute volume of the field batch = volume of water + absolute volumes of cement + sand + coarse aggregate =

$$\frac{142}{62.4} + \frac{282}{3.15 \times 62.4} + \frac{654}{2.65 \times 62.4} + \frac{869}{2.55 \times 62.4} = 13.13 \text{ cu. ft.}$$

The correct unit water content is:

$$\frac{142}{13.13} \times 27 = 292 \text{ lb. per cu. yd.}$$

Using the corrected value for unit water content, as determined by the trial batches, the batch quantities for the adjusted mix having a water-cement ratio of 0.53, may be computed in the same manner as for the trial mix. This procedure should result in concrete having the desired three-inch slump because it is a recognized fact that for a given slump and aggregate gradings, substantially the same unit water content is required for any water-cement ratio.

The adjusted mix for 292 lb. of water per cu. yd. and a 0.53 water-cement ratio is:

1:2.4:3.3.

After the initial adjustment for water-cement ratio has been made and after stabilized operating conditions have been established, the optimum percentage of sand should be determined. This may be done by trial, noting the water requirement for each percentage of sand tried, until the lowest percentage compatible with proper workability, with a safe working margin, is established. The mix is then again adjusted to maintain the correct water-cement ratio.

Rich mixes normally require less sand than lean mixes. After the optimum percentage of sand has been determined for a given mix, cement and sand may be interchanged by absolute volumes without affecting the water content or slump. For practical purposes a change in cement content of one sack per cubic yard of concrete will permit a change in sand percentage of about two per cent.

Changes in consistency may be made by simply increasing or decreasing the amount of water to provide the desired slump, with appropriate adjustment to maintain the specified water-cement ratio.

The job mix should not be adjusted for minor fluctuations in water-cement ratio. A variation of $\pm$ 0.02 (by weight) to maintain a constant slump is considered normal.

Various shortcuts, simplified rules, and charts may be used for making adjustments in concrete mixes with sufficient accuracy for most practical purposes. Since these shortcuts are dependent upon individual conditions and subject to wide personal preference, no attempt is made to describe them in this report.

DISCUSSION

By T. C. POWERS, GUY H. LARSON AND W. H. PRICE

By MR. POWERS†

I have been asked by the chairman to discuss why Committee 613 selected the water-cement ratio as the basis for its recommended method of designing mixes. It is unlikely that among the members agreeing to this basis of design any two would give exactly the same reasons. Therefore, what is set forth below should be considered as the opinion of the writer, with which the other members of the committee concur in varying degrees.

The properties of concrete depend mainly on three factors: (1) The properties of the cement-water paste, (2) the properties of the aggregate, (3) the relative proportions of and the interaction between these two major constituents. Of these three (1) is by far the most important, for with (2) and (3) constant the quality of the concrete can be changed many fold, whereas with (1) constant only relatively small variations can be brought about by variations in (2) or (3) when using aggregates that meet standard specifications and produce workable mixes.

The hardened cement-water paste, which is the binding medium in concrete, can be regarded as a distinct constituent—an entity that can be separated from other ingredients of the mix. The quality of this paste can be controlled at will by the selection of the kind and proportion of cement in the paste and by controlling the conditions under which the reactions of hardening take place. Because this binding medium in concrete is such a distinct, definable material and because its quality is the most important single factor influencing the quality of the concrete, it is logical that the required quality of the paste should be specified in terms as direct as possible. Specifying the characteristics of the cement and the maximum permissible water-cement ratio is the most direct step that can be taken in obtaining the desired quality in hardened cement-paste.

Innumerable tests from both field and laboratory give ample proof of the importance of the water-cement ratio. The underlying cause of

*These discussions, *not* a part of the Committee's report are published on the recommendation of the chairman, Mr. Blanks, as background for subsequent more general discussion—EDITOR

†Assistant to the Director of Research, Portland Cement Association and member Committee 613.

this importance is not difficult to understand. The cementitious hydration-products develop in the water-filled spaces between the original cement-grains. The cementing value of these hydration-products is controlled to a large extent by their density, or compactness, which in turn depends upon the amount of space which they must fill. The amount of space to be filled is fixed at the beginning by the water-cement ratio and therefore this ratio is by far the most important factor regulating the potential quality of the paste. This significant fact alone justifies any procedure that provides direct fixing of the composition of the paste as an alternative to the indirect control provided by procedures dealing primarily with cement-content, consistency, and aggregate-gradation.

It is readily agreed that the selection of the paste does not alone meet all the requirements of a practical method of mix-design. Different results can be obtained from the same paste with different aggregates, and pastes made with different types of cement will have different properties. These important facts have been taken into account in the procedure recommended by the committee. The procedure is such that minimum requirements for the quality of the paste, and for the strength or any other property of the concrete as a whole, may be met by using information from test-results or by conservative estimates based on data given by the committee. Under some conditions it may be necessary to produce concrete having strength greater than the required minimum to provide a paste sufficiently strong and impermeable to withstand weathering. Under other conditions it may be necessary to use a stronger, denser paste than would otherwise be necessary, to produce the necessary compressive or flexural strength. In fact, many considerations may influence the selection of the water-cement ratio, but whatever the basis for selection of the water-cement ratio remains the factor around which the mix is designed.

By MR. LARSON*

Upon reading the first paragraph of the introduction to the "Recommended Practice for the Design of Concrete Mixes," one gathers that the "Practice" contemplates the consideration of all constituents as they affect the concrete before and after hardening. Upon reading farther one notes a certain emphasis on the advisability of making tests to determine the concrete-making properties of the particular materials at hand. But when we get into the actual design procedure, we find the water-cement ratio selected from a table of certain recommended values; no method or procedure for actually determining by test or investigation the water-cement ratio required by given materials to develop a desired

*Wisconsin State Highway Commission and member Committee 613.

quality of hardened concrete is given; and the design procedure concerns itself with designing a workable combination of materials with no further concern expressed over the quality of the hardened concrete. Apparently the water-cement ratio is assumed to be the one factor controlling the quality, strength and durability, of the concrete. This is not a correct assumption, since different aggregates require different water-cement ratios to produce the same quality of concrete.

It has been said that the paste is by far the most important factor in the concrete, and that changes in concrete possible with changes in paste (water-cement ratio) are many times greater than those possible with different aggregates, thus inferring that the aggregates are of minor importance. In this comparison no limit is placed on changes in water-cement ratio while aggregates are restricted to a certain minimum quality. If the water-cement ratio is confined to a range reasonable and economical for the work at hand, then many aggregates meeting standard specifications will be found which, when interchanged, will cause just as great changes in the concrete as will changes in water-cement ratio.

It is agreed, however, that water-cement ratio is very important. And a table of conservative recommended values to use when it is uneconomical or impracticable to make an investigation to determine the water-cement ratio required by given materials to produce concrete of desired quality should be included in the "Recommended Practice." But values in such a table must, of course, contemplate about the poorest aggregates that are likely to be used, and even then are not absolute assurance of proper quality. On the other hand, if it is maintained they should never be exceeded regardless of quality of aggregate, it limits the one who would make a complete design of a mix, including determination of water-cement ratio.

There are ample data and evidence to show that aggregates have a very important bearing and are one of the main factors affecting quality of the concrete. The writer, therefore, believes that a recommended practice for design should include a method or procedure for determining the water-cement ratio required by given materials to produce hardened concrete of the desired quality, as well as a procedure for obtaining a workable combination of materials. With the apparent inadequacy of the present methods for testing aggregates directly, and evaluating the test results in terms of concrete-making qualities, this means actual trial and tests of the concrete.

By MR. PRICE*

The nucleus of the mix-design procedure recommended by Committee 613 is embodied in Table 5, which provides a simple and direct method

*Engineer, Bureau of Reclamation, Denver. Not a member of Committee 613 but an active collaborator with his associates at the Bureau, Messrs. Blanks and Vidal, Chairman and Secretary of the Committee.

for arriving at the percentage of sand and corresponding unit water content for estimating the first "trial mix." Because of possible extensive use of this simple table, the writer believed it desirable to compare the data with findings of various investigators.

The curves plotted in Fig. A and B show that the values tabulated in Table 5 of the committee report for sand of medium grading and rounded coarse aggregate are in substantial agreement with those recommended by others prominent in concrete mix design literature. There is some disagreement, however, regarding the difference between the percentage of sand for mixes made with fine sand and that for mixes in which a medium or coarse sand is used. For the same richness of mix, and within the limits of sand grading shown in Table 5, Dunagan [1] recommends little or no change in the percentage of sand. Kelly[7] and the concrete manual of the Bureau of Reclamation recommend about three per cent less fine sand and three per cent more coarse sand than the percentage of medium sand, whereas Jackson[2] recommends about two per cent for these differences. For concretes of comparable workability, Kellermann[14] found that the required amount of sand having a fineness modulus of 2.75 was about 3.5 per cent more than that of sand having a fineness modulus of 2.22.

Where the gradings of the sand and coarse aggregate are reasonably uniform, an increase or decrease in the percentage of sand will require an increase or decrease in the water content for maintaining the same slump. This increase or decrease in water content, for mixes of about the same richness, amounts to about 2.5 lb. per cu. yd. for each one per cent increase or decrease in sand. But where the grading of the sand changes as from medium to fine, the increased fineness and corresponding decreased sand content may result in little change in water requirement. Kellermann[14] shows practically no change in water content of concrete made with sands of different fineness where the percentages of sand used were those required for maintaining equal workability. The committee report has recognized this by recommending the same water content for the fine, medium, and coarse sands although different percentages of sand are used in maintaining equal workability.

It is generally agreed that angular coarse aggregate requires more sand and water for the production of satisfactorily placeable concrete than does rounded aggregate. Table A, compiled by the writer, shows the increase in percentages of sand and water contents found necessary to produce angular coarse aggregate concrete having the same workability as rounded coarse aggregate concrete. This table indicates that a 4.6 per cent average increase in the percentage of sand and 28 pounds average increase in water are necessary where angular coarse aggregate

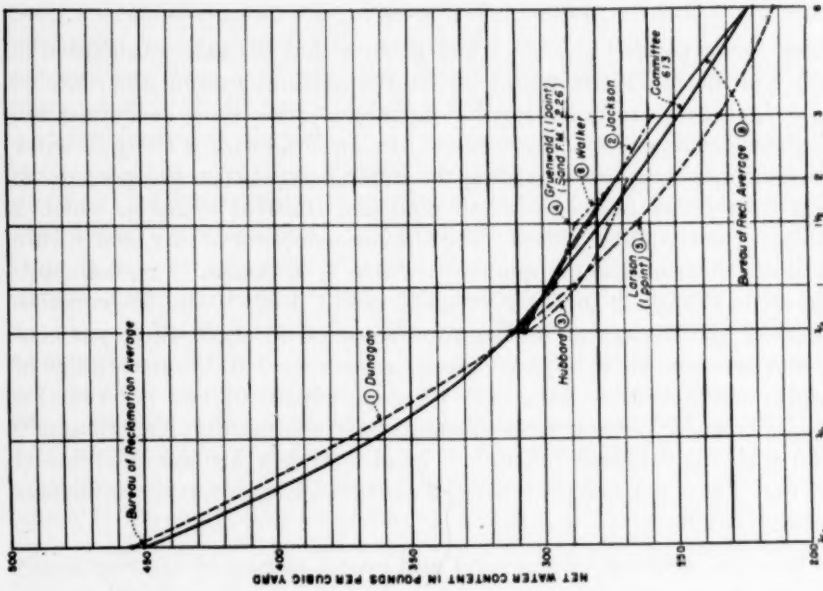

Fig. B—Relationship between water content and maximum size of aggregate

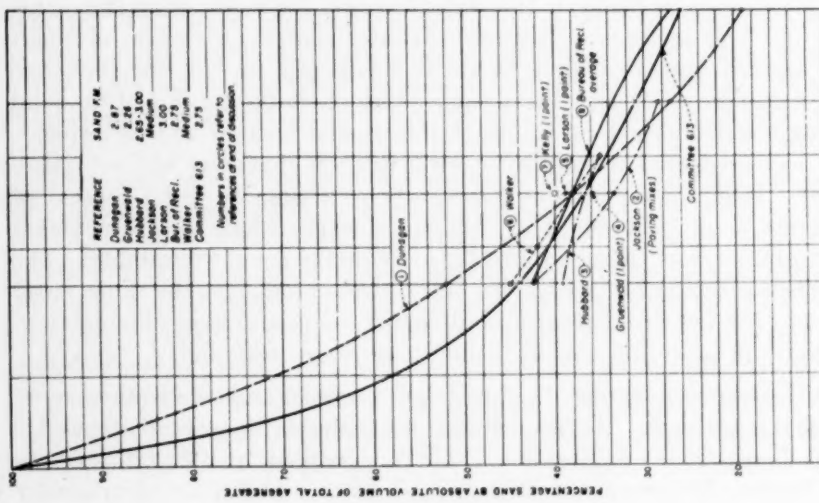

Fig. A—Relationship between percentage of sand and maximum size of aggregate

is employed. The values of 5 and 25, respectively, in the committee report, are in good agreement with this table.

Very little data could be found on the use of crushed-stone sand as compared with natural sand. Kelly[7] recommends a three per cent greater percentage of sand where stone sand is employed and Goldbeck[19] found that crushed sand required about 29 pounds more water than did natural sand to produce mortars of equal consistency. This difference in water requirements should not be as pronounced for concrete mixes and the 15 pounds increase recommended by the committee appears reasonable.

The percentages of sand and the unit water contents recommended in Table 5 of the committee report were found to be in good agreement with the average values mentioned in this discussion and it is concluded that they should be applicable for designing a starting mix with any set of materials where due consideration is taken of the grading and particle shape of the aggregate.

TABLE A—INCREASE IN PERCENTAGE OF SAND AND POUNDS OF WATER PER CUBIC YARD OF CONCRETE FOR ANGULAR COARSE AGGREGATE AS COMPARED TO ROUNDED COARSE AGGREGATE

Reference Number**	Increase in percentage of sand for comparable workability at constant W/C				Increase in pounds of water per cu. yd. of concrete for same slump with increase in sand				Increase in pounds of water per cu. yd. of concrete for same slump and adjusted to no increase in sand			
	Max. Size Aggregate				Max. Size Aggregate				Max. Size Aggregate			
	¾-inch	1½-inch	3-inch	6-inch	¾-inch	1½-inch	3-inch	6-inch	¾-inch	½-inch	3-inch	6-inch
1	2	3	3	2	34	33	37	39	30	26	29	33
2	5	5	5		17	17	17		6	5	4	
3	4	3			18	30			9	23		
	3*	3*			30*	33*			23*	25*		
9		3		8		35		24		28		1
10		7.5		4.5		34		19		16		6
11		5	6	7.5		21	21	20		9	5	−2
12	4				31				22			
13		6				47				33		
14		6				38				24		
15										24		
16										46		
17										11		
18										19		
Average	3.6	4.6	4.7	5.5	26	32	25	26	18	22	13	10
Grand average	4.55				28.3				17.1			

*1-in. and 2-in. maximum aggregate included only in grand average.
**References at end of discussion.

REFERENCES

1. Dunagan—"The Application of Some of the Newer Concepts to the Design of Concrete Mixes," ACI JOURNAL, June 1940; *Proceedings* V. 36, p. 649.
2. Jackson—Unpublished letter to Chairman of ACI Committee 613 dated June 22, 1940. Based on average conditions for pavement mixes.
3. Hubbard—National Slag Association (File 94-13-C). Comparison of angular and rounded coarse aggregates of 2.65 specific gravity at W/C of 0.58 by weight.
4. Gruenwald—"Lean Concrete Mixes," A.S.T.M. *Proceedings* 1939.
5. Larson—Unpublished letter to Chairman of ACI Committee 613 dated Feb. 26, 1940.
6. Walker—"How to Design Concrete Mixtures," ACI JOURNAL, Jan. 1939, *Proceedings* V. 35, p. 211.
7. Kelly—ACI JOURNAL, September 1941; *Proceedings* V. 38, p. 83.

Bureau of Reclamation—Unpublished data.

8. Average of mixes made with gravel aggregates from 41 widely separated projects.
9. Average of trial mixes made with crushed diorite and limestone coarse aggregate from vicinity of Shasta Dam. Values compared to average obtained with gravel coarse aggregate.
10. Average of trial mixes made with crushed dolomite coarse aggregate from vicinity of Buchanan Dam, Texas. Values compared to average obtained with gravel coarse aggregate.
11. Field mixes Altus project. Values compared to average obtained with gravel coarse aggregate.
12. Norris Dam—Tech. Report No. 1. Field mixes with crushed dolomite coarse aggregate and rectangular stone sand. Values compared to average obtained with gravel coarse aggregate.
13. Glanville—Road Research Tech. Paper No. 5. Comparison was made between river and crushed coarse aggregate.
14. Kellermann—"Designing Concrete Mixtures for Pavements"—*The Crushed Stone Journal*, Nov-Dec. 1940.
15. Jackson—"Comparative Tests of Crushed Stone, and Gravel Concrete in New Jersey." *Public Roads*—Vol. 8—1928. Value is average of all mixes with maximum size aggregates ranging from 1 inch to 3 inches.
16. Kellermann—"Effect of Type and Gradation of Coarse Aggregate upon the Strength of Concrete"—*Public Roads*, June 1929—Value is average of 2-inch and 1¼ inch maximum aggregate mixes.
17. Blanks, Vidal, Price, and Russell—"The Properties of Concrete Mixes," ACI JOURNAL, April 1940; *Proceedings* V. 36, p. 433.
18. McMillan—Report of the Director of Research—1928—Portland Cement Association. Value is average difference between comparable limestone and gravel mixes of Series 186 for seven gallons per sack.
19. Goldbeck—A Study of Shape of Particle on the Mortar-Making Properties of Stone Sand—*The Crushed Stone Journal*, Nov.-Dec. 1940.

This report will be formally presented and will be open for discussion at the 38th Annual Convention, but is not now eligible for adoption as an ACI Standard. All discussion of the present report should reach the ACI Secretary in triplicate by Apr. 1, 1942.

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Discussion of a report of Committee 613:

Proposed Recommended Practice for the Design of Concrete Mixes*

CONVENTION DISCUSSION: R. F. BLANKS, T. C. POWERS, GUY LARSON, C. W. BRITZIUS, R. W. HURLBURT, A. C. IRWIN; Since the Convention: J. C. WITT, C. H. FORESMAN, ARTHUR RUETTGERS

By R. F. BLANKS
(Chairman Committee 613)

I believe it was just two years ago at this time that I stood on this very spot and made a promise to the Institute membership that Committee 613 would not let them down. I want to say now that that was not merely an idle boast; I simply did not know any better. Many times since then I have been on the verge of admitting that we could not make good on that promise. However, I can stand before you today and with a clear conscience say that Committee 613 has not let you down.

It is probably true that we have not given you a universally standard method of mix design, as might be inferred from the title of our report, "Proposed Recommended Practice for the Design of Concrete Mixes." To many, the title probably implies that here are all the answers to everybody's problems—a procedure that will fit all jobs and any combination of conditions. I do not believe that it is possible to develop such a standard method for mix design, nor do I believe it is necessary. I do think that in the Committee report we have given you the basic concept of mix design—the basic factors which should control the proportioning of concrete materials. The report constitutes, so to speak, a common language, so that when I talk about concrete mixes and some one else talks about concrete mixes, we will at least be talking in the same general terms. Such a common meeting ground should go far toward eliminating much of the confusion of thought that now exists.

Since the report has been published for some little time, I see no object in reading it now. I might point out a few of the high lights, and also,

*ACI JOURNAL Jan., 1942; *Proceedings* V. 38, p. 193.

a few of the low lights. The recommended procedure is divided into six steps. The first step is the selection of the water-cement ratio. It was a long time before the Committee came to general accord that the selection of the water-cement ratio should be the first step in the design of concrete mixes. Mr. Powers' discussion of the report very clearly sets forth the reasons why that was done. Without going through and reading the several steps as recommended for the procedure of designing concrete mixes, I am going to pass on to the mechanics by which those steps are accomplished.

Under "Selection of water-cement ratio," it says that the water-cement ratio should be selected on the basis of, first, the required durability, and second, the strength specified. The maximum allowable water-cement ratio for the type of construction and exposure conditions involved, may be selected from Table 1 which, as you know, is a time honored and well precedented table. However, the report doesn't tell you what to do with an aggregate or combination of aggregates and cement which under a freezing and thawing test or in actual service will withstand ten cycles, in comparison with an aggregate-cement combination that will stand years of service or an indefinite number of cycles of freezing and thawing. That corner of the report might be considered as one of the low lights. The water-cement ratio required for the specified compressive strength should be determined by laboratory tests. If laboratory tests cannot be made, a table is given which will serve as a general guide for compressive strength-water cement ratio relationship. For flexural strength, there is no substitute for laboratory tests. There is still another kind of strength which is of primary importance, for example, in the design of large dams. That is shearing strength, and perhaps this might be passed off as another one of the low lights of the report.

The nucleus of the entire report is Table 5, which I think is a very useful compilation of many summarized data. I would call your attention to the discussion prepared by Mr. Price and printed along with the report, wherein he gives some of the background for the derivation of Table 5. The unit water contents and percentages of sand listed in Table 5 are weighted averages from a large number of investigations as published or reported by many investigators. It cannot be expected to hold absolutely for every set of materials and conditions, but I believe it will be the exception, rather than the general rule, that wide deviations from the values listed will be found.

The method set up for actually computing the concrete mix after you have made the necessary pre-determinations is on the basis of absolute volume. I would not recommend that you go out on a job and attempt to follow through the detailed sequence of computations in arriving at a

mix to start the work. Several years ago a very pertinent cartoon appeared in the *Saturday Evening Post*. I wish I had a slide of that cartoon but maybe I can describe it well enough so that you can get the idea. It depicted a concrete mixer, with several workmen standing around, leaning on their proverbial shovels. Here was a man with a wheelbarrow of sand, another with a sack of cement, and still another with a concrete buggy, all apparently waiting for something. Very much in evidence in the picture was a man in a white collar frantically feeling through his pockets as though he had lost something. The caption to the cartoon was—"The boss lost the recipe." Now, there are many practical devices or working tools that may be used to avoid the plight of the man in the cartoon. I know that many readers consider the lack of such tools as one of the weak points of the report, which merely says, "various short cuts, simplified rules and charts may be used for making adjustments in concrete mixes with sufficient accuracy for all practical purposes."

The main advantage of the Committee's recommended procedure for the design of concrete mixes, as I see it, is its simplicity. We could add rule after rule, detail after detail, and we could include a discussion of any number of excellent, highly technical methods of mix design that appear in the literature. To do so would result in a report so voluminous that it would defeat its own purpose.

By T. C. POWERS*

Without repeating all of my discussion already published in the JOURNAL, I will discuss the matter of the water-cement-ratio basis of designing mixes. As Mr. Blanks said, the committee did not always agree, and there was perhaps more disagreement over this basis of mix design than over any other one point. I think, however, the degree of disagreement was normal in view of the present state of knowledge concerning these matters. The differences of opinion tended to persist because of the lack of an undeniably fundamental basis on which to settle the questions. There are so many things about the physics and chemistry of concrete mixes that we should know and that none of us does know that it is not surprising that we reach different conclusions from the things we think we know. Different experiences lead to different concepts when knowledge is essentially empirical in character.

At present there is only one fundamental principle that can be applied consistently in designing concrete mixes. About this one thing we can agree: The maximum degree of impermeability, the minimum degree of shrinkage, and the maximum strength that can be developed from a

*Assistant to Director of Research, Portland Cement Association, Chicago; Member Com. 613.

paste of cement and water are limited by the amount of water that is used with the cement. We know also, though perhaps it is not apparent to those who have not been studying the subject in detail, that the cement-water paste is an entity—an individual ingredient of concrete that can be isolated and studied separately. If the cement industry produced the cementing medium that gives concrete its strength, the product would be a paste. The designer of mixes would have to specify in advance the desired quality of paste. The problem of design would not, however, be solved by that step alone, for when the selected paste is combined with aggregate to make concrete, the properties of the resulting mixture will depend upon a great many factors about which we don't know as much as we should.

At the present state of our knowledge, there is no adequate way of specifying how factors other than the quality of the paste are to be taken into consideration except for flexural or compressive strength. As for other properties, frequently of greater importance than strength, particularly durability, the committee could do nothing else than to refer to recommendations based on service records. No test for durability has been sufficiently well standardized to warrant its incorporation in a recommended procedure for designing mixes.

However, we know from experience that if a given water-cement ratio is chosen, the quality of the concrete as a whole can be expected to fall within certain limits. Therefore, the selection of the water-cement ratio is recommended as the starting place. Beyond that, as many refinements may be added as available testing equipment and individual experience will permit.

By GUY H. LARSON*

Being guilty of casting the lone dissenting vote in the final Committee balloting on this recommended practice, the obligation of presenting the minority view, I presume, is mine. The water-cement ratio, to be sure, has an important bearing on the quality of concrete. However, in Wisconsin we have a variety of concrete aggregates and we find that with the same water-cement ratio and the same cement content, the strength of the concrete produced by some of these aggregates differs greatly from that produced by others. We do not know the exact reason for it in each case, but we are convinced that the difference does exist. Some freezing and thawing tests we have made indicate there is also a difference in durability. It appears that the characteristics and properties of the materials as well as the amount of water used per sack of cement, have an important bearing on the quality of the concrete. I therefore think that a recommended practice for designing concrete mixes

*Wisconsin State Highway Commission, Madison; Member Com. 613.

should include a method or procedure for considering all the constituents and their properties and determining the water-cement ratio necessary with the particular materials at hand to produce certain desired results in the hardened concrete, as well as to obtain a properly workable fresh concrete. When given an opportunity to work as a member of Committee 613, it was found one or two other members had similar views, and such a procedure was introduced in a draft of the recommended practice. But other members had what they thought to be equally good reasons for not including such a procedure. When the final draft appeared without it, the recommended practice to me seemed incomplete, and now I find myself in the position of lone dissenter. However, after listening to the contributions to this convention by Messrs. Meissner, Tuthill, Pearson, Goldbeck and Walker, I do not feel so badly about that position. I have enjoyed my committee work and contacts, and feel that I personally have been the chief beneficiary.

By C. W. BRITZIUS*

The Committee's recommended procedure for the design of concrete mixes is very good. I like it because it is based on the fact that for a given set of materials the water content per yard of concrete for a given consistency as measured by the slump test is constant regardless of the cement content.

There has been discussion questioning the desirability of using the water-cement ratio as a basis for design. I believe that the water-cement ratio should be used and that the water-cement ratio should not be picked from an average table, but should be selected on the basis of the strength the materials selected for a job will produce as is determined by test. It is not well understood by the average concrete contractor or architect that for interior concrete, such as columns and slabs, the water-cement ratio selected should be based only on the actual compressive strength produced. If the concrete is to be exposed to weathering or other deteriorating forces, the water-cement ratio should be selected on the basis of the values of Table 1. As has been pointed out additional work remains to be done to determine the relation of water-cement ratio to durability for different materials.

The values of Table 2 that give the strength to be obtained for varying water ratios are in the light of my experience conservative by about 1000 psi at a water ratio of 5 gal., and about 500 psi at 8½ gal. For a given water-cement ratio there are a number of factors that determine the strength of the concrete. These are the cement, the type and grading of the fine aggregate, the type of coarse aggregate, and the properties of the mix, such as sand content and slump. On the basis of my experience

*Twin City Testing Laboratory Inc., St. Paul, Minn.

designing mixes with many types of materials, I have found that there is a wide variation in strength for a constant water-cement ratio. A combination of factors contributing to low strength may cause a 6-gallon water-cement ratio to produce a strength as low as 3000 psi, and again if the materials are conducive to high strength, a strength of 5000 psi may be produced.

I believe that the slide shown earlier of strengths obtained during construction of the PCA Building is not representative of present day construction. You will recall that there was apparently no relation between water-cement ratio and strength. I believe a great part of the lack of uniformity was due to the fact that test specimens were job cured. Concrete control should be judged on the basis of specimens moist cured at 70 F. and the results of job cured cylinders should be carefully used in judging the quality of the concrete.

To illustrate the uniformity that one can expect of controlled concrete, I would like to cite the results of concrete tests of laboratory cured cylinders made during construction of the Farmers and Mechanics Bank in Minneapolis. We designed the mix and inspected the concrete at the central mixing plant. Our tests indicated that a water-cement ratio of 7¼ gal. per sack would produce a concrete testing 3500 psi which was 500 psi over the 3000 lb. designed for. During the concreting operations the water-cement ratio was maintained constant, but the mix varied from 3 to 7 in. slump, 42 to 50 per cent in sand content, and from 5 to 5½ sacks in cement content. In spite of these variations in the mix, the average of the 50 28-day cylinder tests was close to 3600 psi and 90 per cent of the tests were between 3400 and 3800 psi. The highest was 4250 and the lowest was 3200 psi.

Table 5 of the "Recommended Practice" is of special interest to me, for recently I completed a series of tests to determine the effect of the shape of the coarse aggregate particle on the workability of concrete. My determination of workability was made using the slump test, the Powers remolding test, visual inspection, and reference to the ease with which the concrete could be rodded. My investigations show that the extremes of the difference in particle shape require a change of 11 per cent in sand ratio as compared to the 5 per cent recommended by the Committee, which recommendation is substantiated by Table A, page 207. I believe that the values of this table are possibly low because contractors and engineers have become accustomed to the fact that concrete in which the coarse aggregate is crushed rock is difficult to place, and hence do not insist on high workability when using crushed coarse aggregate.

My tests, in which I found that 11 per cent more sand is required with crushed rock than with rounded river gravel, were made using the extremes of coarse aggregates as to particle shape. The rounded gravel was fur-

nished by the Winona Sand & Gravel Co., and this naturally rounded gravel was picked over to remove the little crushed oversize it contained. The crushed rock was furnished by the Trap Rock Co. from its quarry at Dresser Junction, Wisc. This rock is extremely hard, sharp, and angular. The void content of the rounded gravel was 32 per cent and for the crushed rock 42 per cent. The gradation of both gravels was made the same, being of 1½-in. to No. 4, according to ASTM specifications. The cement and sand were the same in both series of tests. The whole investigation was made using a water-cement ratio of 6.5 gal per sack.

Sixteen separate batches were tested for each coarse aggregate. The test procedure was standardized and a mixing machine was used. The test data was plotted in the manner as is recommended by Mr. Powers with the cement content as the ordinate and the sand ratio by absolute volume as the abissa, with values of equal workability plotted thereon.

Our test of the rounded gravel showed that for a remolding effort of 20 jigs the "optimum" sand content was 37 per cent. The slump was 3 in., the cement content was 5.25 sacks per yard, and the water-cement ratio was, as was stated before, constant for all tests at 6.5 gal. per sack.

Our test of the crushed rock showed that for a remolding effort of 20 jigs the "optimum" sand content was 48 per cent. The slump was 3 in., and the cement content was 5.79 sacks per yard. The difference in sand ratios is 11 per cent and the cement contents differ by 0.54 sacks per yard.

Table 5 is accurate for the materials in the Twin Cities area as to the water contents per yard. I believe that this table should have a column under each of the two main headings, "Rounded Coarse Aggregate" and "Angular Coarse Aggregate," called "voids," that would aid in the table's use. It is known that the angularity of the coarse aggregate can be measured by the voids, so why not include the voids so one may obtain the water per yard by interpolation? A variation in grading of the same coarse aggregate within the ASTM limits does not appreciably affect the void content. I also believe that a 1-in. maximum size coarse aggregate should be included in Table 5.

The State of Minnesota Highway Department specifies its mixes for its various types of work on the basis of the void content of the coarse aggregate. By its specification, the two coarse aggregates I studied would require an 8 per cent difference in sand content and a difference of cement of about 0.4 sacks per yard. The water-cement ratio is held constant.

By R. W. HURLBURT*

It is stated that the first step is selection of water-cement ratio and, lacking experience and where durability is not a factor, the designer is

*Engineer of Tests, Canadian Inspection & Testing Co. Ltd., Toronto—Discussion presented in Mr. Hurlburt's absence by R. A. Crysler, Canada Cement Co., Ltd., Montreal.

referred to Table 2. It is definitely stated that the water-cement ratio for the required compressive strength may be taken from the table.

Committee 501 (March-April 1936) recommended a somewhat different table of water-cement ratios for use where trial mixes were not made.

The Canadian Engineering Standards Association has in preparation a specification including a similar table of recommended water-cement ratios.

The comparison between these various tables is shown in Table B.

TABLE B

28 Day Design Strength, psi	Water Cement Ratio by Weight		
	ACI Committee 613	ACI Committee 501	CESA
2000	0.75	0.67	0.66
2500	0.71	0.60	0.59
2800	0.67		
3000		0.53	0.52
3200	0.62		
3500			0.46
3600	0.58		
3750			
4000	0.53	0.44	0.42

In other words Table 2 would permit a design strength of 2800 psi in one case and 4000 psi in another case where the other tables recommend 2000 psi and 3000 psi for the same water-cement ratios.

While it is admitted that the Committee 501 and the CESA are probably overly conservative, I believe that a table promulgated under the auspices of the American Concrete Institute should take into consideration the fact that the information will possibly be used on jobs where aggregates of slightly questionable quality may be used and where there is no definite control or measure of the moisture in the aggregates together with the somewhat unreliable methods of actually batching water.

Taking into consideration the variations and questionable control on a very large number of jobs, which do not warrant laboratory investigations of the concrete making properties of the materials it is proposed to use, I cannot agree with Mr. Larson and the Committee when they refer to Table 2 as "a table of *conservative* recommended values."

By A. C. IRWIN*

I congratulate the Concrete Institute on the excellent work it has been doing. That is no news to you. It has been in existence a long time and

*Dept. of Subways and Superhighways, Chicago.

has existed by virtue of its own momentum, with no professional, or rather no employment reasons why any of you should spend your money and time finding out about concrete. Many of you come here to be refreshed and kept up to date on what is going on. That is also commendable. I believe however that most of you will agree with me that the progress made seems to be tremendously slow. Twenty years ago you were saying pretty much the same thing you are saying today. I don't mean that you have not progressed, because you have made a lot of progress, but maybe what you need is a new direction of your activity. Referring to this report will illustrate what I mean. I note that in the recommended procedure for design of concrete mixes there are six steps, all of which have been presented as steps to take in the reports of the Concrete Institute for many years. No. 2 says "select the limits of slump that will permit proper handling and consolidation of the concrete under the job conditions involved." That is next to impossible unless limits for those job conditions are set. Are you going to allow contractors to use a vibrator that will cause concrete to flow into the forms, or will you limit the use of the vibrator to the purpose of enabling you to employ drier mixes? This body and other bodies that prepare specifications as a guide for others must get some punch into what to do; in other words, you have got to state in more specific terms what you mean when you say "job conditions." I am sure that a number of men here could sit down and write out from their experience that "job conditions" are established entirely for ease and low cost of doing the work without much regard to the quality of the finished product. This committee knows what "job conditions" should be condemned, but unless this is done the contractor has no restraint on him and what you do here is simply stymied at once before you start.

In preparing specifications, engineers and architects go to the Concrete Institute, to the ASTM, to the AREA and so on, and pull out their specifications. They adopt certain portions by reference and copy more—including the mixing and proportioning of concrete as well as control methods. Job control specifications are a great deal more important than some of the detailed specifications you put in. You have got to branch out and state how this work must be done—*must be done.*

Along the same line—we have Tables 1, 2, 3, 4 and 5. By doing specific gravities of course we can compute cement content from Table 5 or the water ratios used for different jobs from another table. Other discussions have brought out the fact that the character of the aggregate has a whole lot to do not only with the workability and ratio of the coarse to fine aggregate but also to the strength because aggregates affect the water-cement ratio necessary for workability. There is a large strength factor of safety in the table that takes care of a great many things but it does not

necessarily take care of durability. One of the discussors referred to specified laboratory soundness tests prior to fixing the water ratio. That is impracticable on most jobs. Tests must be such that they can be done quickly. Ordinarily you cannot tell a contractor just where he shall get aggregates, and if you men would formulate a specification for making tests on aggregate that could be carried out in a few days at most that would tell you whether or not the aggregate was durable under the use exposure, you would have something of great practical value. This would take care of unfamiliar aggregate. Following this, more elaborate tests could be made to establish the character of aggregates from deposits which would lead to a classification of them as to character.

I think you have gone at the present time about as far as you can in the way of the designing of concrete mixes with the information that you have at hand. Then what is the next step? Concentrate on giving the engineer the "tools" necessary to better workmanship.

By J. C. WITT*

Committee 613 has prepared a very good report. Two items, particularly, deserve favorable mention.

The first of these is the basing of the calculations on absolute volumes. It has been found more accurate and more convenient to proportion concrete materials by weight, than by volume. This had led to the preparation of tables showing materials quantities by weight. The assumed specific gravity and moisture content of each aggregate may be shown without emphasizing that the tables are accurate for these conditions only. This has brought about a tendency to overlook the fundamental importance of absolute volumes, as has been pointed out in recent papers.†

The second item is the use of the phrase, "from test data, experience or established relationships." This permits a choice of procedures—which is very desirable.

On page 194, the selection of the water-cement ratio should not be the first item of the procedure. In this sequence, the relation of water to cement is fixed, but the quantity of cement is a function of the other three materials present. This does not eliminate mixes in which the quantity of cement is too low for proper durability. The quantity of cement and the maximum water-cement ratio should be selected first. After adjustment, the water-cement ratio should be as low as conditions permit.

On this basis, a slightly different procedure for the design of concrete mixes is proposed. To facilitate comparison, the same terminology has been employed:

*Technical Director, Marquette Cement Manufacturing Co.
†*Concrete*, 49, 33-9 (1941) and *Rock Products*, 44, No. 12, 75 (1941)

1—Select the quantity of cement and the maximum water-cement ratio from test data, experience, or established relationships, to meet the specified requirements for durability and strength (compression or flexure).

2—Select the limits of slump that will permit proper handling and consolidation of the concrete under the job conditions involved.

3—Determine the largest maximum size of available aggregate suitable for use under the job conditions involved.

4—Estimate from test data, experience, or established relationships, the minimum volume of sand per cubic yard of concrete that will provide the proper degree of workability, and permit the water-cement ratio to fulfill the requirements of 1.

5—Compute the trial-mix proportions that will satisfy the conditions established in steps 1 to 4. Make such adjustments in the trial mix on the job as may be necessary.

The materials quantities may be found readily by the methods described in the papers to which reference has been made, when either the original procedure or the suggested modification is followed.

By C. H. FORESMAN*

The design of a concrete mix is the focal point of our knowledge of concrete. This operation, if properly carried out, collects the various properties of the primary materials and blends them into characteristics, which should be predetermined, of the plastic and hardened concrete. These predetermined characteristics are not merely the slump and a hoped-for minimum strength; they include many other qualities equally essential. Naturally, the degree of refinement necessary depends on the service requirements, but a method that does not permit of control of all of the characteristics required is of doubtful value as a standard or recommended practice.

That the water-cement ratio is an important consideration in the design of concrete mixes is almost axiomatic. It does not follow, however, that nothing else need be considered. It is well to remember that the ratio between the water and the cement determines the strength of concrete only if other conditions remain the same. The proposed method makes no provision for the maintenance of equality in the "other conditions."

The design of concrete mixes is properly divisable into two sections; the first having to do with the quality and physical properties of the primary materials and the second part with the mechanics of combining the primary materials to produce characteristics in the resulting concrete as determined by the preliminary investigations and the service require-

*Concrete Engineer, The Board of Public Education, Pittsburgh, Pa.

ments of the job. The report of Committee 613 is concerned only with the second part—the mechanics of mix design, so there is no object in discussing the preliminary investigations, except that they must produce two very important results. The first of these results is the water-cement ratio for the cement and aggregates being used. The report of the Committee clearly states that the determination of the W/C ratio is a laboratory procedure but it would seem that it could be even more emphatically stated that the figures shown in the W/C ratio tables should be used only in an emergency or where the work is of a minor nature.

The other important result of the preliminary investigation, as it affects the design of the mix, is the grading of the combined aggregate. A great deal of work has been done on the grading of concrete aggregates and there are numerous and varied opinions about it but one fact cannot be disputed: Uniformity of results in concrete cannot be maintained unless the grading of the combined aggregate is constant. The report does not mention the grading of the combined aggregate and the proposed method does not provide for the maintenance of uniformity in such grading.

In connection with the maintenance of uniformity in the grading of the combined aggregate, it should be mentioned that when only two primary aggregates are used such uniformity can be maintained only when the specifications require the use of aggregates graded within very narrow limits. Three, four or five sizes of aggregate should be used, depending on the maximum size permissable. When three or more aggregates are used, the trial mix method can be used only with difficulty and should be manipulated by a designer with considerable experience. Even then, it fails to produce results which justify the use of more than two aggregates. At a time when more and more large users of concrete are demanding the separation of coarse aggregates and in many cases fine aggregates as well, it scarcely seems suitable to propose a method which, for all practical purposes, is limited to two aggregates.

Where only one fine and one coarse aggregate are used, the trial mix method is probably as good as any but it seems rather pointless to set it up as a standard and publish a description of it as such. It has been adequately described in any number of publications, at least some of which are available to Institute members merely for the asking.

To persons familiar with the trial mix method, the procedure outlined in the report seems unnecessarily complicated. Bearing in mind that the object of the method is merely the production of a workable concrete with a definite water-cement ratio, the procedure can be confined to the following very simple steps:

1. Mix air-dried fine and coarse aggregates in the proportions, by weight, selected (the proportions may be set by the designer's own experience or by using one of the many tables available).

2. Add cement and water in the proper ratio until the desired consistency is obtained.

3. Measure the yield.

4. Determine the cubic yard quantities by multiplying the weights of materials used in the trial batch by 27/yield.

If the mix is too sandy or too harsh, do just what the name of the method indicates—try it again, varying the proportions of fine and coarse aggregates as necessary.

If volumetric measurements are desired, weights can be translated into volumes in the field with proper allowances for bulking due to moisture.

Moisture content of the aggregates should, of course, be added to the weights of the aggregates when weight measurements are used and, in any case, should be deducted from the amount of mixing water. Incidentally, the determination of the moisture content of the aggregates is not just a laboratory test; it is a routine field operation.

Since this discussion has termed the proposed method unsuitable, it might be thought appropriate for the writer to produce an alternate method. The writer, however, feels that the proven ability of the committee members is such that their efforts should produce a method superior to the one he uses. At the same time he suggests that the requirements for any method proposed as a standard include the following:

1. It should be adaptable to multiple aggregate design.

2. It should provide for the control of as many of the properties of concrete as is humanly possible.

3. It should provide for the maintenance within very narrow limits of a constant grading of combined aggregates, without imposing too narrow limits on the grading of the primary aggregates.

It is not the intent of this discussion to suggest that the use of the trial mix be barred or that it should not be recognized as a legitimate method of design. Under certain circumstances and for certain purposes it is a very useful tool and certainly a brief description of the method should be included in the report of the Committee but why should the American Concrete Institute adopt a method of such limited scope as a standard?

By ARTHUR RUETTGERS*

Considering the maze of data published on concrete mix design and allied subjects in the past few years, it is refreshing to find a design procedure as systematic and clear cut as that formulated by Committee 613. The Committee is to be commended for its contribution.

After reviewing the report critically, the writer has been impressed with one apparent weakness; namely, that the utility of the report has been restricted, perhaps unnecessarily, by too abbreviated a treatment of two phases of the subject.

Consider first the question of aggregate grading. Aside from providing specific procedures for determining the maximum size of aggregate and for estimating or establishing by test the minimum percentage of sand to be used—procedures which are admittedly important and useful—the report has little to offer on grading, either in respect to the gradings of individual aggregate size fractions or their combinations. Detail grading information needed in effecting the procurement and utilization of aggregates to the best advantage must therefore be sought elsewhere or by test. On page 199 the report states:

> The Committee recognizes that there are many excellent methods for proportioning fine and coarse aggregate for trial concrete mixes. Where conditions or personal preferences dictate, use may be made of any of the basically sound methods described in the current literature.

However, the report does not reveal, for the benefit of the reader, the nature or identity of the methods recognized.

What seems to be needed, if the present price of ignorance is to be reduced, is some system along the lines proposed by Weymouth,† whereby one may judge the suitability of any combination of separated aggregates in any proposed concrete mixture and may, at the same time, determine at least the direction in which optimum grading lies. The problem assumes greater importance with increase in maximum size of aggregate, since the larger the number of aggregate fractions, the more numerous are the combination possibilities, and the greater is the disadvantage of having to explore such possibilities in the laboratory or to await field trial.

The writer is aware of specification grading limits such as those prepared by the American Society for Testing Materials and the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, but these grading specifications leave much to be desired and do not encompass the range of aggregate sizes dealt with in the report of ACI Committee 613. Certainly the membership of Committee 613 is particularly qualified to tackle the problem.

*Bureau of Reclamation, Denver, Colo.

†C. A. G. Weymouth, "Effects of Particle Interference in Mortars and Concretes," *Rock Products*, Feb. 25, 1933, p. 26.

Consider now the section of the report under the caption "Computation and Adjustment of Trial Mix."

The absolute volume relationship embodied in the computation procedure illustrated on pages 200 and 201 of the report makes the procedure easy to grasp and to retain. This is a valuable feature. On the other hand, one needs only to perform the 36 indicated calculations, plus the 21 calculations not specifically indicated but needed for completion of the example, to realize that the detail procedure is ill-adapted to field use—at least to the extent that it pertains to mix adjustment. The distribution of the 57 calculations (a single calculation representing a multiplication, division, addition or subtraction) is as follows:

	No. of Calculations
(1) Computation of trial mix proportions	14
(2) Computation of trial batch weights	7
(3) Determination of actual water-cement ratio	4
(4) Computation of correct unit water content	11
Subtotal	36
(5) Recomputation of mix proportions and batch weights for corrected unit water content	21
Total	57

The writer is not advocating the elimination of the procedure approved by the Committee but he does recommend that the Committee reconsider its position (as expressed in the last paragraph of the report) and supplement the procedure with a few judiciously selected working tools which, taken together, will have more or less universal appeal.

One time-saving and error-minimizing device whose merit has been proven by several years' experience is a nomographic chart shown in Fig. C. An explanation of the use of the chart follows—the illustration being the one employed in the Committee report.

EXAMPLE OF USE OF NOMOGRAPHIC CHART

Given: Water-cement ratio = 0.53 by weight.
Unit water content = 305 lb. per cu. yd.
Proportion of sand = 42 per cent.
Specific gravity: Sand, 2.65; coarse aggr., 2.55.

To find: (A) Mix proportions for trial batch.
(B) Adjusted mix proportions after trial has shown water ratio to be 0.50 instead of 0.53.

Solution: Part A.

Place a transparent straightedge across the nomograph—see line (1) on chart—to connect scale points for water ratio of 0.53 and water content of 305.

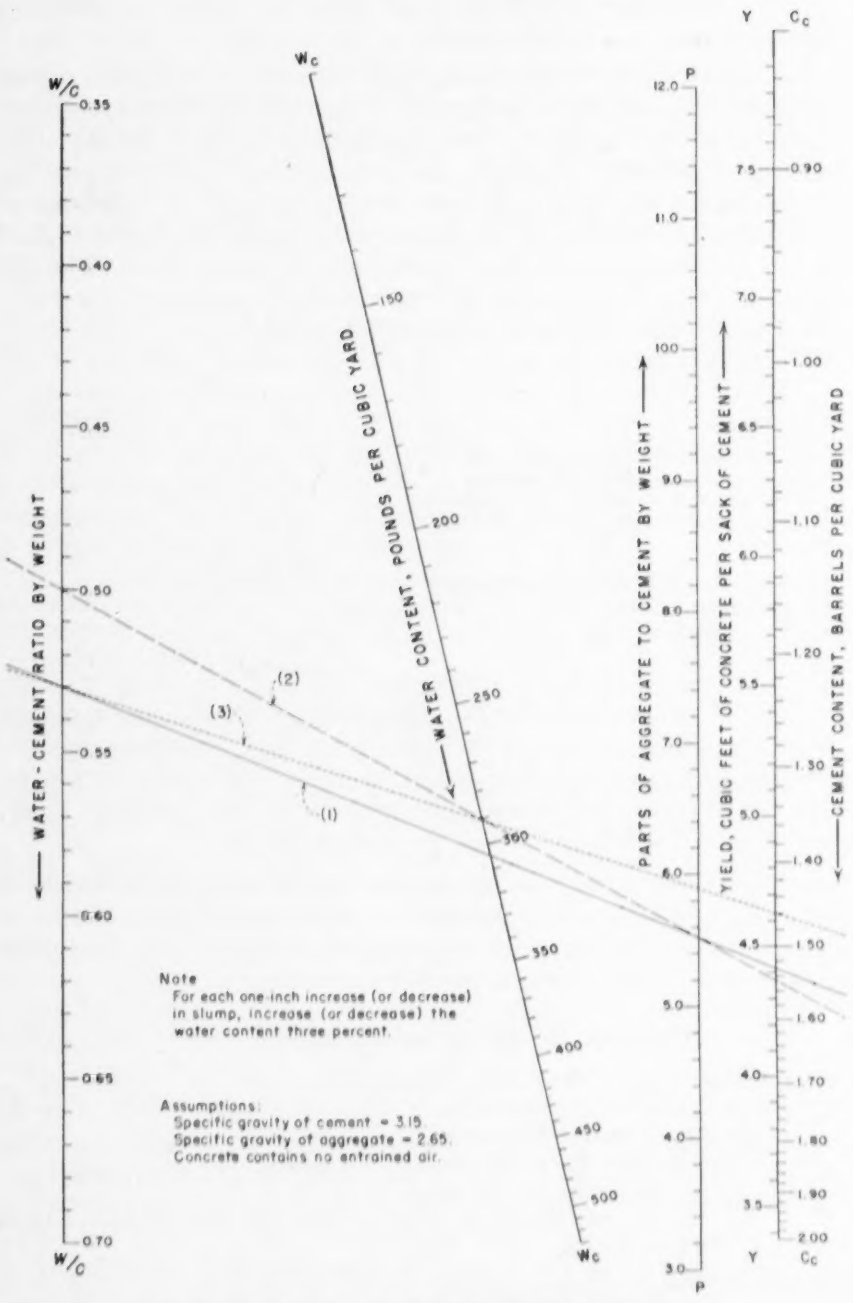

Fig. C—Chart for design and adjustment of concrete mixes

Read off chart: Parts of aggregate = 5.5 (for 2.65 sp. gr.)
Cement content = 1.53 bbl. per cu. yd. (6.12 sacks per cu. yd.)
Parts of sand = 42 per cent of 5.5 = 2.3.
Parts of coarse aggregate = 5.5 − 2.3 = 3.2 (for 2.65 sp. gr.)

$$= \frac{2.55}{2.65} \times 3.2 = 3.1 \text{ (for 2.55 sp. gr.)}$$

Trial mix = 1 − 2.3 − 3.1 by weight.

Solution: Part B.

Connect scale points on nomograph—line (2) on chart—for water ratio of 0.50 and parts of aggregate of 5.5 (2.65 sp. gr.)
Read off chart: Water content = 292 lb. per cu. yd.
Connect scale points on nomograph—line (3) on chart—for water content of 292 and water ratio of 0.53.
Read off chart: Parts of aggregate = 5.9 (for 2.65 sp. gr.)
Cement content = 1.45 bbl. per cu. yd. (5.84 sacks per cu. yd.)
Parts of sand = 42 per cent of 5.9 = 2.5
Parts of coarse aggregate = 5.9 − 2.5 = 3.4 (for 2.65 sp. gr.)

$$= \frac{2.55}{2.65} \times 3.4 = 3.3 \text{ (for 2.55 sp. gr.}$$

Adjusted mix = 1 − 2.5 − 3.3 by weight.

Manifestly, even with devices such as the nomograph, there are still many producers of concrete for whom the system of mix design and adjustment outlined in the Committee report is too involved. This situation could be relieved in a measure by setting up an auxiliary condensed system built on a similar pattern. Thus, since the concrete would generally involve aggregate of ¾ to 1½ in. in maximum size, and of average specific gravity, grading and particle shape, the trial mix could be based on an average unit water content of say 36 gal. per cubic yard, an average sand factor of 40 per cent, and a fixed unit weight of concrete of 4,000 lb. per cu. yd. Also, as the cement would be measured in sacks, the water-cement ratio would, for convenience, be expressed in gallons per sack. The procedure for arriving at dry batch weights would then be similar to that proposed by Walker*, as illustrated by the following example:

Given: A 10-S mixer and a required water-cement ratio of approximately 6 gal. per sack.
To find: The dry batch weights for a trial mix.

Solution:

Estimated water content = 36 gal. per cu. yd.
Estimated cement content = 36/6 = 6 sacks per cu. yd.
Use a 2-sack batch, which would yield about 2/6 cu. yd. of concrete and weigh approximately 4000/3 or 1333 lb.

The dry batch weights would be:

Water = 36/3 or 12 gal. at 8⅓ lb. = 100 lb.
Cement = 2 sacks at 94 lb. = 188 lb.

*Stanton Walker, "How to Design Concrete Mixtures," ACI JOURNAL Jan. 1930; *Proceedings*, V. 35, p. 211.

Total aggregate = 1333 − (100 + 188) = 1045 lb.	
Sand = 40 per cent of 1045 lbs.	= 418 lb.
Gravel = 1045 − 418 lbs.	= 627 lb.
Total	1333 lb.

The approx. mix would be 1:418/188:627/188 = 1:2.2:3.3 by weight.

Assume now that when the concrete is brought to the desired consistency, the water needed is only 11 gal. instead of 12 for the 2-sack batch, which corresponds to a water-cement ratio of 5½ gal. per sack instead of 6. It follows that the only change required in the previously computed batch weights is to increase the weight of the aggregates so as to use up the full allotment of water. As the consistency of the concrete and grading of the aggregates are not to be changed, the new aggregate batch weights can be determined by taking account of the fact that the weights of the total dry materials (cement plus aggregates) are directly proportional, for all practical purposes, to the water-cement ratios—that is:

$$\frac{\text{New weight of dry materials}}{\text{Original weight of dry materials}} = \frac{\text{New W/C}}{\text{Original W/C}} = \frac{6}{5\frac{1}{2}}$$

Original weight of cement plus aggregates = 188 + 1045 = 1233 lbs.

$$\text{New weight of dry materials} = \frac{6}{5\frac{1}{2}} \times 1233 = 1345 \text{ lbs.}$$

New weight of aggregates alone = 1345 − 188 = 1157 lbs.
New batch weight of sand = 40 per cent of 1157 = 463 lbs.
New batch weight of gravel = 1157 − 463 = 694 lbs.
New mix = 1:463/188:694/188
= 1:2.5:3.7.

The writer has long felt that an assembly of simple rules, in conjunction with a few key mixes of known characteristics, may be used to advantage in the design and adjustment of concrete mixes that will serve most purposes. A simplified procedure employing such rules and key mixes is presented in the form of a series of questions and answers. Although comparisons are made with data in the Committee report, the writer is not proposing the procedure as a substitute for the Committee procedure. Rather he is presenting it primarily for the benefit of field men who are looking for an expeditious yet comprehensive and reliable procedure which is not dependent on tables, charts, or similar working data. The simplicity of the various adjustments facilitates maintenance of the water-cement ratio at the maximum safe level, thereby enabling the production of concrete of increased uniformity and economy. It also reduces the need for laboratory tests or for refinements in arriving at initial mixes for field trial.

A SIMPLIFIED PROCEDURE FOR CONCRETE MIX DESIGN AND ADJUSTMENT

Q. 1. What is the principal feature which differentiates the simplified procedure from the procedure in the Committee report?

A. The simplified procedure deals directly with mix proportions rather than with unit water content and absolute volumes.

Q. 2. If the unit water content is not dealt with directly, what is used as a starting point for arriving at a trial mix designed to meet certain requirements?

A. There is an easy-to-remember key mix for each of four maximum sizes of aggregates, as follows:

For	¾ in.	max. aggregate	1-2.5-3.5 by wt.
For	1½ in.	" "	1-2.5-4.0 " "
For	3 in.	" "	1-2.5-5.0 " "
For	6 in.	" "	1-2.5-7.0 " "

These key mixes apply to concrete of 3-in. slump and 0.57 water-cement ratio, made with average natural aggregates (2.65 specific gravity) and sand of medium fineness. It will be noted that only the parts of coarse aggregate differ, and that the successive differences form a geometric series with a ratio of two. Curiously enough, this series relation applies also to the successive maximum sizes of aggregate.

Q. 3. Why was the odd water-ratio figure of 0.57 by weight chosen for the key mixes?

A. Partly because 0.57 is a good working value for a limiting water ratio of 0.60, and partly because the water ratio of 0.57, in combination with a 3-in. slump and appropriate percentages of sand, results in the simple mix proportions stated.

Q. 4. How do the unit water contents and percentages of sand for the key mixes compare with those in Table 5 of the Committee report, which table is likewise based on a slump of about 3 in.?

A. For sand of medium fineness they happen to be virtually the same.

Q. 5. How can the key mixes be used to apply to fine sand or coarse sand?

A. With fine sand, the parts of sand may be reduced and the parts of gravel increased by 0.2. With coarse sand the procedure would be reversed. Thus a 1-2.5-3.5 mix with medium sand would become 1-2.3-3.7 for fine sand and 1-2.7-3.3 for coarse sand. The new percentages of sand would then be in substantial agreement with the values in Table 5 of the Committee report. For medium fine or medium coarse sand, a change in parts of 0.1 should serve.

Q. 6. What compensation would be made for the partial or complete substitution of crushed for natural rounded aggregates?

A. The mix proportions would be modified by merely reducing the parts of coarse aggregate. This single step has two effects; it increases the percentage of sand and also the content of cement paste, thereby tending to maintain the workability of the concrete and the water-cement ratio. The proper reduction to be made in the parts of coarse aggregate would naturally depend on the relative amount of crushed aggregate used and on the extent to which the shape and surface condi-

tion of the aggregates as a whole depart from those of all-natural aggregates. For results approximately equivalent to those obtained by employing the increases in percentage of sand and unit water content indicated in Table 5 of the Committee report, the parts of coarse aggregate in the mix would be reduced about 15 per cent in the case of all-crushed coarse aggregate and 25 per cent in the case of all-crushed fine and coarse aggregate.

Q. 7. How are given mix proportions adjusted to make them apply to concrete having the same slump but a different water-cement ratio?

A. The requirement for such an adjustment is of frequent occurrence. It arises in the determination of a trial mix from a key mix when the water-cement ratio desired is other than 0.57. It also arises whenever it is desired to alter the water-cement ratio of a mix in use, as when a trial mix brought to the proper consistency fails to result in the correct water ratio. The adjustment in mix proportions is accomplished by changing the total mix parts (sum of parts of cement and aggregates) in proportion to the change in water-cement ratio, without changing the grading of the combined aggregates. The procedure is illustrated by the following example:

Given: 1-2.5-3.5 mix, with 0.60 W/C and 4½-in. slump.
Required: Mix for 0.54 W/C and same slump.

Solution

Total original mix parts = 1 + 2.5 + 3.5 = 7.0
Proposed reduction in W/C = 0.06/0.60 = 10 per cent
Required reduction in parts of aggregate (or total mix parts) = 10 per cent of 7.0 = 0.7 parts or say 0.3 parts sand and 0.4 parts gravel.
New mix = 1-2.2-3.1.

Note: It might be explained that the procedure outlined is not without sound support. By considering that the unit water content is unaltered by moderate changes in water-cement ratio when slump and aggregate grading are maintained, it can be demonstrated that the following relation obtains:

$$\frac{(\text{Required parts of aggregate}) + S_a/S_c}{(\text{Original parts of aggregate}) + S_a/S_c} = \frac{\text{New W/C}}{\text{Original W/C}}$$

wherein S_a/S_c is the ratio of specific gravities of aggregate and cement—ordinarily about 0.84. This equation can be simplified to advantage, without introducing appreciable error, by considering the ratio S_a/S_c as unity and therefore as representing cement in the mix proportions. The equation then becomes:

$$\frac{\text{Total new mix parts}}{\text{Total original mix parts}} = \frac{\text{New W/C}}{\text{Original W/C}}$$

Finally, the relation may be expressed in more usable form as:

$$\text{Change in parts of aggregate} = \frac{\text{Change in W/C}}{\text{Original W/C}} \times (\text{Total original mix parts})$$

This is the relation used in the previous example.

Q. 8. How are given mix proportions modified to provide for a change in slump without changing the water-cement ratio?

A. For each one-inch increase (or decrease) in slump, the total mix parts are decreased (or increased) by 3 per cent, without changing the grading of the combined aggregates. Thus to increase the slump of a 1:2.5-3.5 mix by 2½ in. and maintain the water ratio of say 0.55, the total mix parts of 7.0 would be decreased by 7½ per cent, or about 0.5. This would mean a decrease of 0.2 in the sand and 0.3 in the gravel, with a resulting approximate mix of 1-2.3-3.2.

Note: In justification of the procedure outlined, let us suppose that the increase in slump of 2½ inches is brought about without altering the 1-2.5-3.5 mix proportions—that is, by simply using more water. This means that if the extra water required is 3 per cent for each one-inch increase in slump, in accordance with the average value generally assumed, the water ratio of 0.55 will be increased by 7½ per cent or to 0.59. The problem then is to determine the reduction in mix parts necessary to restore the water ratio to 0.55. By using the procedure previously explained in the answer to question 7, the reduction in mix parts is $\frac{0.04}{0.59} \times 7.0$, or approximately 0.5. From this discussion it should be apparent that an adjustment for change in slump without change in water ratio is closely related to an adjustment for change in water ratio without change in slump.

Q. 9. Is there a simple means of adjusting mix proportions to provide for a change in the relative amounts of sand and gravel and still maintain both the water-cement ratio and slump?

A. Yes. For a change of each 0.1 in the mix parts of sand, the parts of gravel are changed in the opposite direction in an amount as follows:

For	¾-in.	maximum	size	aggregate—0.25
For	1½-in.	"	"	" —0.3
For	3 -in.	"	"	" —0.35
For	6 -in.	"	"	" —0.4

To illustrate, suppose that a mix of 1-2.4-3.4 having 1½-in. maximum size aggregate and the required water ratio and slump appears to be oversanded and that it is desired to determine the optimum proportion of sand (giving least cement content for necessary workability) without appreciably affecting the water ratio and consistency in use. A set of mixes that may be tried successively will then be:

1-2.3-3.7
1-2.2-4.0
1-2.1-4.3

Note: There is nothing mysterious about this adjustment procedure if one stops to analyze it. If, for instance, a 1-2.4-3.6 mix is changed to 1-2.5-3.5 (total parts of aggregate the same) and the water-cement ratio is maintained, it is obvious that the only alteration is an increase in the fineness and surface area of the combined aggregates; hence the only effect would be a reduction in slump of the concrete. However, such slump loss would be eliminated by further reducing the parts of coarse aggregate to the point where the tendency for reduction in slump is compensated by the relative increase in cement paste content. Naturally the larger the maximum size of aggregate, the lower

is the surface area per unit weight of coarse aggregate, and the greater is the required change in parts of coarse aggregate to compensate for a unit change in parts of sand. The tabulated changes in parts of coarse aggregate for 0.1 change in parts of sand apply to well-graded, average natural aggregates.

Q. 10. What is the effect of a departure in specific gravity of the aggregate from the assumed average value of 2.65?

A. The solid space occupied in a batch of concrete by aggregate of 2.65 average specific gravity may be maintained for aggregates of lower (or higher) specific gravity by reducing (or increasing) the mix parts of aggregate in proportion to the departure in specific gravity. Thus for 4.0 mix parts of coarse aggregate, based on a specific gravity of 2.65, the parts of coarse aggregate of 2.58 specific gravity would be $\frac{0.07}{2.65} \times 4.0$ = about 0.1 less, or 3.9.

Note. A specific-gravity adjustment in mix proportions is generally an unnecessary refinement, because:

(1) It takes about 0.10 departure in specific gravity to cause a change of 0.1 in the mix parts of sand, and it is only very rarely that ordinary sands suitable for concrete fall outside the range in specific gravity represented (2.55 to 2.75).

(2) For the ordinary run of concrete, the figure for the total mix parts of coarse aggregate is usually not over 4.0. As shown by the above example, this allows a departure of about 0.07 in specific gravity before the parts of coarse aggregate are affected as much as 0.1. Perhaps two-thirds of the suitable gravels and crushed rocks used for coarse aggregate have specific gravities within 0.07 of 2.65.

(3) No matter how determined, trial mix proportions which in practice would hit the mark within the probable error introduced by ignoring normal departure in specific gravity would be considered very exceptional by the writer. Moreover, the indicated process of adjustment after the mix is tried and found wanting automatically takes care of almost any departure in specific gravity.

Q. 11. When cement content is desired, is it also determined directly from mix proportions rather than by absolute volume computations?

A. Yes. The following empirical equation, derived as indicated in Fig. D and giving results which are usually accurate within one or two per cent, is employed:

$$\text{Cement content in bbls. per cu. yd.} = \frac{\text{10 plus one-tenth of total mix parts}}{\text{Total mix parts plus water-cement ratio}}$$

To illustrate, consider the two mixes used in the answer to Question 7. These mixes are

(Mix 1) 1-2.5-3.5 W/C = 0.60
(Mix 2) 1-2.2-3.1 W/C = 0.54

and, by applying the equation, the cement contents are

$$\text{(For Mix 1)} \quad \frac{10 + 0.1(7.0)}{7.0 + 0.60} = \frac{10.7}{7.6} = 1.41 \text{ bbl. per cu. yd.}$$

$$\text{(For Mix 2)} \quad \frac{10 + 0.1(6.3)}{6.3 + 0.54} = \frac{10.63}{6.84} = 1.56 \text{ bbl. per cu. yd.}$$

Note: The reason for selecting the specific mixes used and for computing the cement contents of both is to permit the demonstration, by example, of a second method of

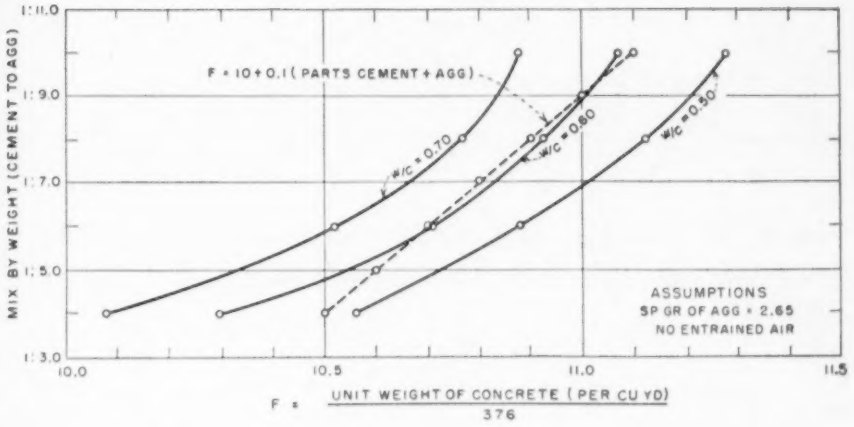

Fig. D—Effect of mix parts and water-cement ratio on unit weight of concrete

computing cement content under certain conditions. The method is based on the principle that when a mix, such as mix 2, is derived from another mix, such as mix 1, to effect a change in water ratio without change in slump or aggregate grading (see Question 7 and answer), the cement contents are inversely proportional to the water ratios. In other words the cement content for mix 2 could have been obtained as follows:

$$\frac{0.60}{0.54} \times 1.41 = 1.56$$

Q. 12. If the simplified procedure outlined were applied to the example used in the Committee report, what would be the nature of the computations for arriving at the proportions of the trial mix and the adjusted mix?

A. The computations would be as follows:

The key mix is 1-2.5-4.0 W/C = 0.57 Slump = 3 in.

For W/C = 0.53, the reduction in parts of aggregate = $\frac{0.04}{0.57} \times 7.5 = 0.5$

The revised mix is 1-2.3-3.7

For coarse aggregate of 2.55 sp. gr., the reduction in parts = $\frac{0.10}{2.65} \times 3.7 = 0.1$

The revised mix is 1-2.3-3.6

For crushed instead of natural coarse aggregate, the reduction in parts is 15 per cent of 3.6 or 0.5

The *trial mix is 1-2.3-3.1.*

The net water-cement ratio found by trial is 0.50 instead of 0.53.

To restore the water ratio to 0.53, the increase in parts of aggregate = $\frac{0.03}{0.50} \times 6.4 = 0.4$.

The *adjusted mix is 1-2.5-3.3.*

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California Experience with the Expansion of Concrete Through Reaction Between Cement and Aggregate*

By THOMAS E. STANTON
Member American Concrete Institute
O. J. PORTEP, L. C. MEDER, and ALLEN NICOL†

SYNOPSIS

This group of papers brings up to date the work done by the Materials and Research Department of the California Division of Highways during the last three years in the investigation of the cause of the serious deterioration of concrete in certain areas of the State.

Many data are presented supporting the contention that the cause of the trouble is a chemical reaction between alkali in the cement and some mineral in the aggregate.

Long time (up to three year) test results are included and methods and results of various accelerated test procedures are described.

Studies on the subject of permissible alkali content, effect of puzzolanic admixtures, the nature of the reactive aggregates, possible correctives and the need for a comparative petrographic study of various known and unknown reactive aggregates are discussed at length.

Interesting data are presented showing the extent to which a highly reactive mineral is apparently innocuous when present in excess quantities which may be as low as only 10 or 15 per cent of the aggregate particles.

Eight lines of investigation are suggested for future research.

By MR. STANTON‡

INTRODUCTION

The nationwide occurrence of instances of the deterioration of portland cement concrete structures through a reaction between some mineral ingredient or ingredients in the aggregates and alkali in the cement, is evidenced by the numerous articles published on the subject during the last two years.

*Received by the Institute Dec. 1, 1941.
†All on the staff of the Materials and Research Department of the California Division of Highways.
‡Materials and Research Engineer.

The earlier reported concrete failures in California were confined to the area along the coast from Monterey to Los Angeles County. The major cause of concrete distress in this area was definitely traced to a reaction between opaline silica in the chert portion of the aggregates and alkali in the cement.*

Although the reported California failures were confined to a portion of the coastal area, laboratory evidence was presented indicating the probable above normal expansion which might be predicted with sand from the San Joaquin River near Friant.

Some of this sand was included in the original test series, being identified as sand No. 27 (Fig. 1). Petrographic analysis shows this sand to be composed chiefly of granite, schist and other rock fragments of quartz, feldspars and biotite mica.

At 36 months expansion up to 1050 millionths of an inch per inch or 0.105 per cent has been observed in 1 x 1 x 10-in. mortar bars of a 1:3 mix using Friant sand and high alkali (1.10 per cent) cement.

This development offers a possible explanation for some of the distress observed in structures built by the Southern California Edison Co. in the development of the Big Creek Power Project in the Sierras above Friant.

Within recent months considerable distress of a nature similar to the map cracking identified with the reaction between cement and aggregate has been observed in the concrete in the Belmont Traffic Circle just west of the underpass at the southeast corner of Roeding Park, Fresno (Fig. 2). Investigation disclosed that in the construction of this concrete pavement by the City of Fresno in 1932, Friant (San Joaquin River) aggregates were used. The cement reported as having been used, was a high alkali cement similar to one of the cements used the same year by the California Division of Highways in the construction of a section of pavement west of Auburn, Placer County and on the Bay Shore Highway test pavement south of San Francisco.

Analysis of the retained sample of state-used cement of this brand shows an alkali content of 0.96 per cent. A neutral aggregate was used in both the Auburn and San Francisco projects and the concrete shows no distress after nine years.

While the deterioration of the Fresno traffic circle concrete has undoubtedly been under way for some time, the extent and nature of the trouble was not brought to the attention of the writer until comparatively recently.

Adjacent to the deteriorated traffic circle project and constructed the same year (1932) is an underpass with connected pavement in which

**Proceedings of the American Society of Civil Engineers, December 1940 and September 1941.

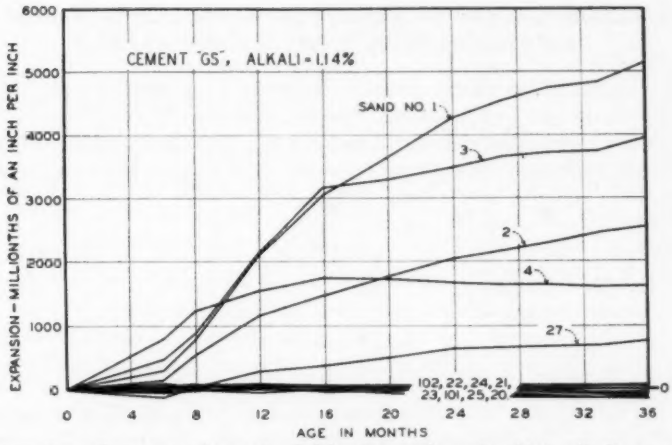

Fig. 1—Expansion of mortars of certain California sands and a high alkali cement. See Table 1 for sand identification.

Fig. 2—Showing condition of concrete on two projects both constructed in 1932 at the Belmont Underpass and Traffic Circle, S. E. corner of Roeding Park, Fresno, California. The aggregates in both projects came from the same source (San Joaquin River near Friant). Left, low alkali cement. Right of joint, high alkali cement.

Picture taken in October, 1941. The same brands of cement with non-reactive aggregates were used the same year (1932), before and after the Fresno work, in the construction of Sections of State Highway pavement on the Bay Shore Highway Test Project north of South San Francisco, San Mateo County without ill effects.

the same aggregates, but a low alkali cement were used. The low alkali cement concrete shows no distress.

DISCUSSION OF PROPOSED ACCELERATED TEST PROCEDURES

The long delayed expansion of the Buck Hydroelectric Plant concrete reported by Kammer and Carlson[1] emphasizes the necessity for the development of some accelerated test procedure which at the same time will not be open to the objection of being so severe that it does not truly represent ultimate field weathering probabilities.

[1]ACI JOURNAL, June 1941; *Proceedings* V. 37, p. 673.

TABLE 1—IDENTIFICATION OF SANDS FOR WHICH LONG TIME (36 MONTH) EXPANSION RESULTS ARE SHOWN IN FIG. 1

Group 1. In this group rock fragments consist chiefly of granitic rock and sandstone with less than 15 per cent quartzite, rhyolite, andesite, siltstone, and limestone, and from 4 per cent to 15 per cent shale and chert. The mineral grains consist chiefly of quartz and feldspar, with less than 10 per cent of pyroxene, amphibole, mica, magnetite, hematite, limonite, serpentine, and calcite. Sands No. 1 to 4 in this group were secured from the following places:

Sand No.	Source
1	Santa Clara River at Saticoy, Venture County
2	Sisquoc River at Sisquoc, Santa Barbara County
3	Piru Creek at Santa Clara River, Ventura County
4	Oro Fino, tributary to Salinas River, Monterey County

Group 2. Rock fragments in Group 2 consist chiefly of granitic rock and gneiss in Southern California, and of graywacke (sandstone) in Northern California, with less than 10 per cent of basalt, andesite, quartzite, and jasper and usually less than 1 per cent of shale and chert. Mineral grains in Group 2 are similar to Group 1. Sands No. 20 to 27 in this group were found in:

Sand No.	Source
20	Coyote Creek, Santa Clara County
21	Russian River, Healdsburg, Sonoma County
22	Livermore Valley, Alameda County
23	Near Olympia, Santa Cruz County
24	Roscoe, San Fernando Valley, Los Angeles County
25	Irwindale, San Gabriel Valley, Los Angeles County
27	San Joaquin River at Friant, Fresno County

Group 3. The sands in Group 3 were made by crushing ledge rock to the desired gradation. Sands No. 101 and 102 in this group were found in:

Sand No.	Source
101	Crushed hornblende diorite, Logan, San Benito County
102	Crushed quartz, Jackson, Calaveras County

Relation of temperature to test results

Carlson, Blanks and others have suggested storing the specimens under temperature conditions ranging from 100 to 170 F. Undoubtedly temperature accelerates the reaction. It would appear desirable, however, to hold the temperature down to not over 130 to 140 F. if criticism of this test procedure is to be avoided. Higher temperatures are subject

to the objection that they may set up reactions not encountered in actual service.

Sodium hydroxide solubility test

It is comparatively easy to break down some unreactive rock particles in a sodium hydroxide solution. This test, therefore, seems to be out at least under the conditions found in California were less than one per cent of the more reactive mineral or minerals is normally present in the fine aggregate. Although these reactive particles are in themselves highly soluble their volume relation to the entire aggregate is so small as to make the test too sensitive for general application, particularly when the other 99 per cent more or less of normally unreactive particles may consist of only slightly soluble minerals, the loss of which in the aggregate may greatly outweigh the total loss of the much smaller percentage of highly soluble reactive particles.

Petrographic analyses

Petrographic analyses offer a fertile field for investigation but unfortunately such analyses are normally practicable only with the larger coarse aggregate particles and even then it may be difficult to isolate the rock types which are the cause of the trouble, particularly when they constitute only one to two or three per cent of the total volume. However, once an aggregate has proved reactive as determined by expansion tests, it is usually possible to identify the probable reactive rocks and then to determine the particular mineral in the rock which is the source of the trouble. The identification of the reactive mineral may be of academic interest, but is of little value as a routine test procedure if we must first determine the reaction characteristics of the aggregate by some other procedure. Petrographic analysis will only be of value if we can develop supporting data connecting certain rock types with invariable reaction to a detrimental extent when present in sufficient quantities. From our present knowledge of the subject, this seems impracticable as evidenced by the experience with the Parker Dam Andesite and Rhyolite, etc.*

Fluorescence

Here again we have a proposed test which as at present understood is too erratic for dependence, if it has any virtue at all. California experience with this test is discussed at greater length in the A. S. C. E. paper.

*H. S. Meissner "Cracking in Concrete Due to Expansion Reaction Between Aggregate and High Alkali Cement as Evidenced in Parker Dam," ACI JOURNAL, Apr. 1941; *Proceedings* V. 37, p. 549.

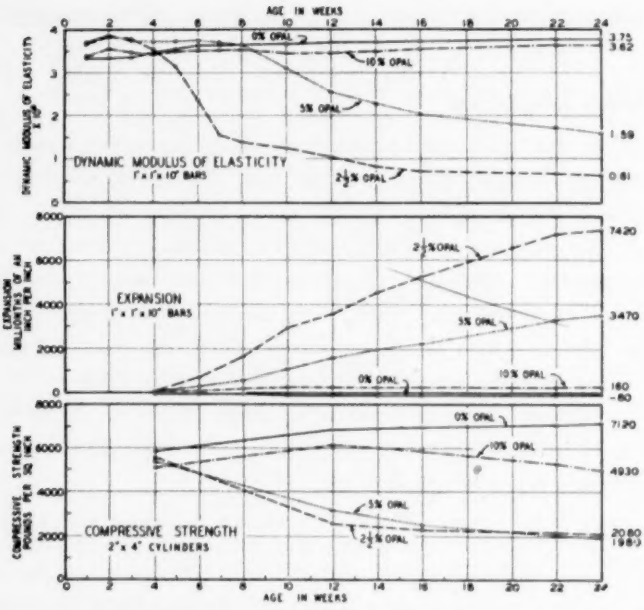

Fig. 3—Dynamic modulus of elasticity, expansion and compressive strength of 1:2 mortar specimens. High alkali (1.14 percent) cement, "GS" + Russian River sand + 0 to 10 percent of thirty to eighty mesh opal from dredger dumps near Roseville, California. All specimens cured in sealed containers.

Modulus of elasticity by dynamic method

It has been suggested that it might be possible to detect any undue reaction at an earlier period through the dynamic modulus of elasticity method than through expansion measurements.

To test the value of this suggestion a number of 1 x 1 x 10-in. mortar bars were fabricated containing 2½, 5, and 10 per cent opal.

It was known from previous long time expansion tests that 2½ per cent of the type of opal used would produce excess expansion in a short period and that 5 and 10 per cent would produce successively less expansion, thus affording an opportunity to determine the comparative merits of the expansion vs. modulus of elasticity methods of test in a short time.

The results of this test series up to 24 weeks are well illustrated in Fig. 3. It will be noted that while there is definite evidence of falling modulus of elasticity at three weeks, the decrease is so slight as to be unreliable as a control test. In only one case (2½ per cent opal) had the elastic modulus lowered appreciably even in six weeks. On the other hand, in six weeks the expansion results had assumed a very definite pattern which became more marked at each successive measurement

period. At the same time, however, there was no increase in the elastic modulus of any of the opal treated specimens after two weeks, except to a slight extent in the case of the least reactive 10 per cent opal content specimens, whereas the specimens to which no opal had been added have shown a steady increase throughout the entire period to date.

As most reactive sands contain smaller percentages of reactive mineral and are, therefore, ultimately less reactive than any of the synthetic sand tests reported above, the dynamic modulus test does not appear to be a dependable substitute for length measurements insofar as an accelerated test is concerned.

PERMISSIBLE ALKALI CONTENT

The earlier California studies indicated that 0.5 per cent maximum alkali content would be safe insofar as objectionable expansion in a comparatively short period was concerned and California specifications at first limited the alkali content to this amount.

Subsequent tests indicated that 0.60 per cent alkali apparently might be tolerated in the California brands of cement, therefore, although it was realized that the lower the alkali content apparently the better the cement insofar as expansion reactions are concerned, the permissible limit was increased to 0.60 per cent in order not to work an undue hardship on those cement companies which might find a lower percentage difficult to comply with without a substantial increase in manufacturing cost.

Later test data indicate that it may be desirable to return to the original limit of 0.5 per cent or even less with some aggregates. One brand of California cement containing 0.67 per cent alkali has been found excessively reactive in a comparatively short period. There has been another instance where even a 0.45 per cent alkali cement mortar has developed disturbing expansion in 32 months under certain conditions of test. A 0.51 per cent alkali cement mortar has developed expansion of over one-tenth of one per cent in 32 months using a standard commercial sand from Ventura County. (Fig. 4).

SUMMARY OF KNOWN FACTS AND SUGGESTIONS FOR FUTURE RESEARCH

Known facts

1. Deterioration of concrete results in certain cases through a reaction between some mineral constituent in the aggregate and the alkalies in cement computed as $Na_2O + K_2O$.

2. The intensity of the reaction is related to the alkali content of the cement (Fig. 4) and likewise to the percentage of reactive mineral in the aggregate increasing up to some "pessimum" amount of reactive

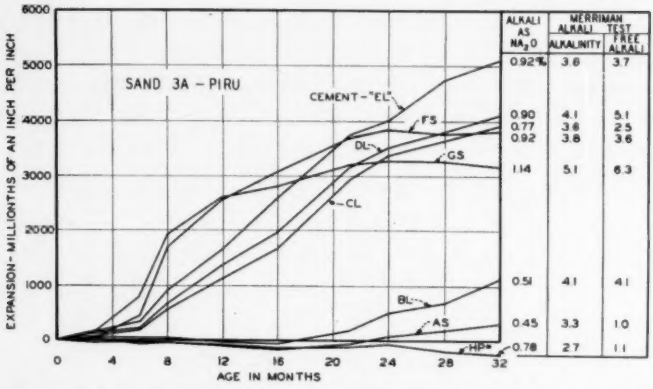

Fig. 4—Expansion of sand cement mortars as influenced by the type of sand and alkali content of the cement. 1:3 plastic mortar mix. Specimens cured in sealed containers. Cement "HP" is a puzzolanic type cement.

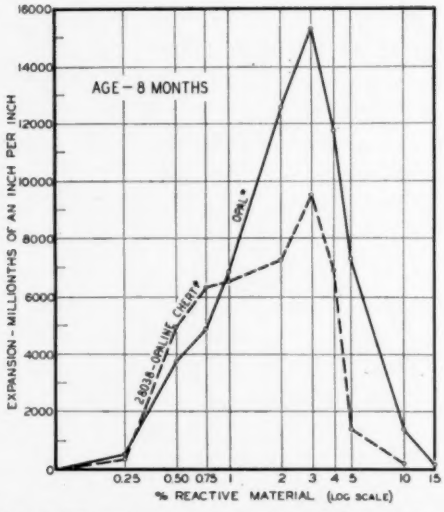

Fig. 5—Expansion effect of different percentages of two high reactive rocks. Rock particles substituted in percentages shown for an equivalent percentage of neutral Russian River Sand. High alkali (1.14 per cent cement used throughout. 1 x 1 x 10-in. mortar specimens, mix 1:2.

Opal secured from dredger dump near Roseville, Calif. †Rock 28038 has been identified as one of the constituents of Ore Fino sand and was secured from ledge deposits found in Monterey County.

aggregate particles and thereafter decreasing to the point where the reaction is such as to produce no excess expansion products of a nature or intensity sufficient to cause rupture (Fig. 5).

3. Opal has been definitely proven to be at least one highly reactive mineral and with one exception is an essential part of all California rocks identified as reactive.

TABLE 2

Comparative expansion in millionths of an inch per inch of 1" x 1" x 10" mortar bars of a 1:3 mix of a highly reactive sand and cements of varying alkali content. Data are arranged in order of relative expansion at 8 months. Figures also show extent to which order of expansion in some cases was reversed at 32 months. See Fig. 4 for further details.

Cement	Alkali Content Percent	Expansion in Millionths in. per in.	
		8 Months	32 Months
GS	1.14	1910	3190
FS	0.92	1660	3810
EL	0.92	892	5040
DL	0.90	645	4060
CL	0.77	590	3900
BL	0.51	27	1100
AS	0.45	–40	350

4. Other rocks not definitely identified as containing opal have been found reactive in combination with high alkali cements of certain brands.

Suggestions for future research

1. Development of suitable accelerated test procedure to determine reactive characteristics of any coarse or fine aggregates. Procedure should standardize on the cement, particularly with relation to the nature and per cent of alkali; also the cement content of the mix and storage temperature.

2. Extent and under what conditions different size aggregate particles are objectionably reactive.

3. Determination of exact nature of the reactive mineral or minerals. In the case of opal which has been definitely proven reactive, ascertain if all opals are equally reactive and if not the composition of the most active type.

4. Difference in reactive characteristics, if any, of the sodium and potassium in cement and the most reactive form; i. e., water soluble, etc.

5. Reactivity of $Ca(OH)_2$.

6. Extent to which silica or puzzolana admixtures in cement affect the reaction and the relative merits of different admixtures of this type. (Fig. 6).

7. Other possible correctives.

8. Determination of the duration of activity of the reaction factors and probable maximum intensity. (Fig. 4).

In connection with the last suggestion, it is interesting to observe the trend of the reactions illustrated in Fig. 1 and Fig. 4. In the test series the results of which are illustrated in Fig. 1 the sand (No. 4) which showed the greatest reaction in 8 months (0.12 per cent) reached a peak in 16

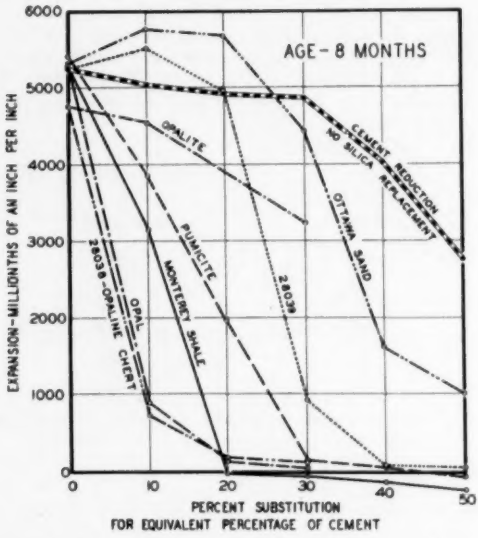

Fig. 6—Effect on expansion through substitution of different powdered rocks, for an equivalent percentage of cement in an expansive mortar composed of high alkali (1.14 per cent cement, Russian River sand and 10 per cent graded siliceous magnesian limestone No. 28039. All substitutions ground to pass a 325 mesh screen. 1 x 1 x 10-in. mortar specimens; basic mix, 1:2.

to 20 months (0.17 per cent), whereas sand No. 1, which developed considerably less expansion in 8 months (0.07 per cent) had developed a much higher expansion (0.52 per cent) at 36 months and was still expanding. A similar condition is noted with relation to the different cements when the sand type is kept uniform (Fig. 4). Cement GS (1.14 per cent alkali) which showed the greatest expansion in 12 months (0.26 per cent) peaked at 24 months at 0.33 per cent whereas cement EL (0.92 per cent alkali) has passed GS and is still expanding at 32 months (0.51 per cent).

By MR. PORTER*

The conditions accompanying early failure in 1932 and 1933 of a concrete pavement (constructed 1927) on the Coast Highway in Ventura County, California, called the attention of the writer to the possible action of alkali in the soil on the cement and aggregate.

A gel, later identified as an alkali aggregate reaction, was observed. Although the trouble at that time was attributed to alkali, the source was assumed by the writer to be the subgrade soil rather than the cement. It was reasoned that sodium sulfate from the soil might have reacted with calcium hydroxide in the concrete to form calcium sulfate and

*Senior Physical Testing Engineer, California Division of Highways.

sodium hydroxide. While it is now evident that the high alkali cement used on the project could fully account for the alkali aggregate reaction it is possible that alkali from the soil contributed to some extent toward the early and complete disintegration of the pavement in this case. The possibility of a similar action with a low alkali cement should not be eliminated from consideration, therefore, where the soil, water or aggregate contains water-soluble salts of sodium or potassium.

It is believed that two cases involving extensive popouts, expansion and cracking in the basements of large buildings in Los Angeles can be laid to the presence of water soluble sulphates including sodium sulphate in the ground water and thus to an aggregate-alkali reaction. In these basements extrusions of gel are numerous and expansion is pronounced as indicated by both popouts and cracking. On the other hand, the condition of the concrete is good on the outside walls above the ground water table as well as in the interior columns and walls. Although the alkali content of the cement in these two basements is not known it seems logical to assume that the trouble is not due to a high alkali cement, otherwise, some evidence of reaction should be apparent in the massive walls and columns above the ground water table.

Following the Ventura pavement trouble in 1933, study was directed to the source of alkali in other pavements and structures. Sodium carbonate had been found as an incrustation on failed structures where the source of the sodium was restricted to some ingredient of the concrete mix and it was reasoned that the sodium was in the form of sodium hydroxide in the concrete prior to carbonation on coming in contact with the air. The water and the aggregates used in concrete pavement and structures showing distress had been subjected to tests for alkali and water soluble salts during construction, but it was known that many of the minerals in the aggregate contained sodium.

BASE EXCHANGE

Many speculations regarding the availability of the sodium or potassium present in some of the shales, cherty shales and feldspar found in the sand and gravel were studied and attempts were made to liberate the sodium by base exchange reactions.

The base exchange reaction was investigated by treating the finely powdered aggregate with distilled water to eliminate all soluble lime compounds and then with a normal sodium chloride solution followed by washing until the filtrate no longer showed a reaction for chlorides. The sample was next treated with a calcium chloride solution and again washed with distilled water until the filtrate no longer yielded indications of lime after which the material in the filter was treated with a normal sodium chloride solution and the filtrate tested for calcium compounds.

In the finely pulverized aggregate so treated only a slight indication of base exchange was indicated.

Several hundred specimens, hand selected to represent the various types of suspected rocks and minerals, were also placed in calcium hydroxide solutions which were protected from carbonation. Observations were carried on for over a year, but all the specimens showed no indications of reacting in the calcium hydroxide solution.

Although these considerations are very interesting and merit additional study, any attempt to account for the alkali through base exchange from the aggregate does not satisfy the California problem as the same aggregate has in many cases been used with both good and bad results, the latter always when the reactive type of aggregate is used with a high alkali cement. Many pavements and structures constructed 10 to 20 years ago with aggregates from the same sources, but with cement from sources manufacturing a product normally low in alkali, are still in good shape If a serious reaction can eventually result from the presence of sodium and potassium in the silicate minerals of these aggregates when used with low alkali cement, very long time tests will be required as much construction of this type shows no excessive expansion or cracking after a score or more years of service.

AGGREGATE

Although the presence of opal in the relatively small percentage of chert and shale particles in the California aggregate can account for the high alkali cement aggregate reaction in all but one case (Friant sand), the very active attack of sodium hydroxide on silica and silicate minerals in general makes it easy to comprehend the many similar failures now reported elsewhere which on first glance bring under suspicion many of the igneous and metamorphic rock types that have been used extensively as aggregate throughout the world. It is reasonable to assume that the action is largely due to the presence of some specific reactive mineral or minerals rather than to the particular rock types as such. At least it is hard to conceive that the unaltered essential minerals of granite, rhyolite and andesite are causing the trouble with the high alkali cement. If this were so, the widespread use of such aggregates would make it necessary always to use a low alkali cement to safeguard the work. It is much more likely that weathering and leaching have altered the minerals of the suspected reactive igneous aggregates or that some accessory or secondary minerals are included. As an indication of one possibility, vesicular basalt intrusions near the coast of California contain some calcite, zeolites and opal. It is believed that these secondary minerals were deposited in the vesicles or small gas holes since the rock solidified.

Another possibility is that the weathering of some igneous rocks results in microscopic remains of opal or some other silica or silicate minerals readily soluble in a caustic solution on or near the surface of the rock particles. It is not unusual for weathering, particularly arid conditions, to result in complex coatings or incrustations on the surface of aggregate particles of any petrologic type present in the area. The "caliche" type gravels commonly found in the desert regions of California, Arizona and other western states are an outstanding example of complex weathering and precipitation. While the "caliche" coatings contain a large amount of calcareous material they may also include clay, gypsum, soluble silica, alkali and many other compounds and impurities resulting from weathering and precipitation. It is clear that aggregates carrying such complex surface coatings should be thoroughly studied in connection with high alkali cement-aggregate reaction. Such coatings may not only contain reactive minerals, but may also contribute additional alkali tending to accelerate the expansive reaction. Before the problem of high alkali cement-aggregate reactions can be answered, it will be necessary to develop tests to readily and positively detect the presence of reactive minerals.

CHERTS

Cherts have been the subject of much concern for many years and many investigators have studied their suitability for use in concrete. Much confusion of the subject results from the fact that chert is not a definite type of either rock or mineral and varies from soft, light weight, porous material to hard, dense, flint-like particles approaching quartz in density and hardness. Furthermore, cherts range in composition from the hydrous silica opal to chalcedony and with or without various inclusions and impurities. The studies made in eastern states until recently have centered on the durability of cherts under conditions of freezing and thawing. California studies of recent years have concentrated on the effect of chert in concrete at moderate temperatures, but with high alkali cements. Both types of cherts (opal and chalcedony) are common in California, but investigations clearly indicate that the excessive expansion with a high alkali cement is due to the opaline type and never to the pure chalcedonic type.

During an extensive trip through Illinois, Michigan, Indiana, Missouri and Kansas in the summer of 1939, the writer inspected many of the gravel deposits containing chert and also obtained specimens of some of the types of cherts found in the limestone quarries and gravel deposits. This inspection trip plus identification tests on typical samples secured indicate that cherts commonly found in the limestone quarries consist largely of chalcedony; however, small inclusions of opal were found in

the ground mass of some specimens. Considering the probable origin of the chert in mid-western limestone formations, it appears probable that some opal may be included in many of these deposits. Whether the opal is ever present in sufficient amount and in a form readily available for reaction with a high alkali cement requires additional study.

The sand and gravel in Michigan, northern Indiana, and northern Illinois contain a considerable amount of chert which has been deposited along with many other igneous and metamorphic rock types by glacial drift. The chert particles vary widely in color, density, texture, luster, etc., but it is probable that chalcedony is the predominating mineral. Many of the pieces, however, pop and split apart when heated in a flame, suggesting the presence of hydrous silica. Petrographic studies indicate the presence of opal in some of the gravel particles obtained from the area. Considerable study would, of course, be required to determine if an appreciable amount of the cherty gravel contains opal, as the glacial drift containing chert extends over a large area and, furthermore, the original source of the material extends back over the course of the glacier for a great distance. Gravel from the Kaw River in Kansas consists largely of chert. Some of the particles apparently contain opal. It was of considerable interest to the writer to find that some gravel deposits on US-50 between St. Louis and Jefferson City, Missouri, which had been transported by streams flowing north from the Ozark Mountains, consist largely of chert particles.

Tests reported by Walker and Bartel* show a relatively high solubility in sodium hydroxide for chert from commercial sources (presumably eastern and midwestern) which also suggests the presence of opal or some other form of silica more soluble than pure chalcedony.

In view of the trouble experienced with chert in concrete in eastern and midwestern and southern states and the fact that opal is quite often found associated with chalcedony, it will be of interest to learn whether the map-cracking and excessive expansion of concrete containing chert in these sections is associated with the use of high alkali cement. An examination of such concrete for the presence of sodium silicate "gel" associated with the typical high alkali cement-aggregate reaction in California might assist in clarifying the problem.

CONCLUSION

From the standpoint of the safe acceptance of aggregate for possible use in important concrete construction with high alkali cement, the problem becomes more complex from day to day as trouble similar to that observed in California is reported from many widely separated sections of the country. The types of rock and minerals already impli-

*Stanton Walker and F. F. Bartel, "Discussion on the Soundness of Chert", Proceedings of 43rd Annual Meeting A. S. T. M., 1940, Vol. 40, p. 1044.

cated by the failures reported are usually included in some form or other in most sand and gravel deposits. Examination of the aggregates involved clearly indicates that the usual type of analysis, including petrological classification and standard tests, as well as any special tests now available and suitable for rapid acceptance determinations, are entirely inadequate to preclude the use of aggregates containing reactive minerals other than opal. Aggregates which in many respects are similar to the material present in construction now reported to be affected by the high alkali cement-aggregate reaction are undoubtedly still being used. It is, therefore, imperative that the reactive mineral or minerals in each case be positively identified at an early date. In the absence of this information or a reliable rapid test for the reactivity of the aggregate with high alkali cement, many aggregates which may subsequently prove to be entirely suitable must be questioned until a service record or suitable accelerated or long time expansion results with high alkali cement are obtained. The problem is of special importance in many western states where commercial aggregates with long service records are not economically available for construction in many localities and it is necessary to continually develop aggregates from new and unproven local deposits for major bridge, highway, and dam construction projects.

By MR. MEDER*

STUDIES TO DEVELOP CORRECTIVES

There is no doubt but that the expansion in the California case is caused by a reaction between hydrous silicates soluble in the presence of a strong base, and the alkalis in the cement. It seems reasonable to assume that these alkalis are converted to alkali hydroxides by the hydration of the cement, and are then in a position to act upon any reactive material available.

Throughout the study of the alkali-aggregate reactions, they have been assumed to be and therefor spoken of as somewhat ordinary chemical reactions between the alkali constituent of the cement and hydrous silica, or opal in the aggregate. These reactions do not take place under the usual chemical conditions where there is free mobility of the reacting components. In concrete there is a very limited amount of free water, insufficient even to moisten a paper when pressed against a freshly broken surface. It is under such a practically dry condition, with solutions that are probably very concentrated, that these reactions are progressing. It is interesting to note that even in the presence of a very limited amount of water, there is sufficient accumulation of moisture at any point of reaction to produce wet areas, and even to form drops of fluid that exude

*Assistant Physical Testing Engineer, California Division of Highways.

down the sides of the specimens. This water is undoubtedly concentrated from a large area surrounding the reactive particles.

Under the study of correctives, one thing should be kept in mind. The reaction that causes the expansion is somewhat tied up with the development of a form of sodium silicate. According to some authorities sodium silicate, when painted on concrete surfaces, for hardening purposes reacts with the calcium hydroxide released by the cement to form a calcium silicate and sodium hydroxide. If this reaction actually takes place there is no reason why the same action should not take place in the presence of the sodium silicate formed by the reactions of the alkali and the opal, thereby resulting in the continued regeneration of $Na(OH)$. If such is the case, then the only possible corrective would be one which would convert at least some of the alkali into a harmless form, preferably an insoluble compound thereby reducing the alkali content of a cement to a point where there is insufficient to develop expansive forces of sufficient magnitude to disrupt concrete.

Correctives

Possible correctives may be divided into groups: (a) cement treatments and small additions designed to alter the alkalies into modified or harmless salts, and (b) siliceous or puzzolanic admixtures.

It was felt that if the alkali hydroxides could be converted to chlorides, the chlorides, being neutral, would not react upon the soluble silicates. To this end, bars were fabricated with the high (1.14 per cent) alkali cement, a reactive sand, and 0, 1½, and 3 per cent $CaCl_2$. The $CaCl_2$ failed to reduce the expansion.

Somewhat later, similar attempts were made to reduce the expansion by the use of ferric and aluminum chloride. These salts were used in 25 per cent, 50 per cent, 100 per cent, 150 per cent, and 200 per cent of the equivalent of the alkali oxides in the cement. The theory was that through the reactions

$$FeCl_3 + 3Na(OH) \text{——} Fe(OH)_3 + 3NaCl$$

or

$$AlCl_3 + 3Na(OH) \text{——} Al(OH)_3 + 3NaCl$$

the highly basic compounds that were apparently causing the expansion might be altered to such an extent as to be rendered harmless. As in the case of $CaCl_2$, the $FeCl_3$ and $AlCl_3$ failed to reduce the expansion, but on the contrary materially increased it.

Since all of the attempts to reduce expansion by converting the alkali hydrates to chlorides failed so completely, it was decided next to determine the effect of sodium and potassium chloride added to the mortar. Sets of specimens were fabricated using high and low alkali cement and a neutral sand to which had been added 10 per cent of the reactive rock No. 28039. Sodium and potassium chlorides were added in amounts

from 0 to 10 per cent by weight of the cement. Briefly, the results at 90 days were as follows: with high alkali cement but no *NaCl* addition, the expansion was 0.66 per cent. When *NaCl* was added, the greatest expansion (0.90 per cent) was with the 0.5 per cent addition. The expansion then fell off slightly with increased percentages of *NaCl* dropping to 0.47 per cent with 10 per cent addition. With the low alkali cement, there was no expansion without the *NaCl*, but with 0.5 per cent the expansion was 0.13 per cent and with 2.5 per cent it was 1.07 per cent or considerably greater than with the high alkali cement, a phenomenon which requires further investigation.

Similar results were obtained with potassium chloride except that the magnitude of expansion to date has not been as great.

A study is now being made of the effect of *NaCl* and *KCl* upon cement. It appears, from very sketchy data now available, that the basicity of the cement is increased considerably by the addition of the chloride, about 25 per cent more acid being required for a titre than with the plain cement plus water.

The next step in the search for a corrective was the addition of a finely divided opaline material which it was thought might react with and consume the alkalies before they could noticeably attack the coarser sand particles. Accordingly, specimens were fabricated containing 0, 1 and 3 per cent (by weight of the cement) of diatomaceous earth. This, likewise failed to reduce the expansion, but rather increased it.

One of the early suggestions offered to explain the expansion was the possible reaction between magnesium carbonate in the siliceous magnesian limestone No. 28039 and sodium hydroxide to form a hydrated sodium carbonate which would have a much greater volume than the total volume of the reacting components. It was felt that if this were the cause of the expansion, then finely powdered $MgCO_3$ might react with the hydroxide in the early plastic stages, thereby reducing the ultimate expansion.

Tests with $MgCO_3$ as an admixture likewise failed to reduce the expansion, but rather tended to increase it slightly in most cases.

A sodium alginate was next tried. When 3 per cent was added by weight of the cement, the expansion was reduced by 50 per cent, but was still 0.2 per cent at 8 months. Smaller amounts of the sodium alginate had practically no effect.

Next in order was vinsol resin. When used in the very low proportions in which this resin is customarily used, it had practically no effect, but when used in much greater proportions (0.5 and 2.0 per cent) a large amount of air was entrapped thereby inducing a large void space. The

higher percentages reduced the expansion noticeably, but they also slowed the set and seriously reduced the strength.

According to an article in the *Industrial and Engineering Chemistry*, Industrial Edition, Vol. 33, No. 5, May, 1941, Page 692, sodium sulphate was added to cement to increase its resistance to sulphate action. Although it was not felt that this admixture would help in this particular problem, because as can be seen from the following reaction

$$Na_2SO_2 + Ca\ (OH)^2 \text{———} 2NaOH + CaSO_4$$

the sodium hydroxide would theoretically be greatly increased, nevertheless it was decided to make a few tests along this line. As was expected the specimen which contained 4 per cent of sodium sulphate by weight of the cement showed very high early expansion.

So far, the only material other than siliceous admixtures, which has proved effective in reducing expansion has been aluminum powder. this powder reacts with the alkali as follows

$$2Al + 2NaOH + 2H_2O \text{———} 2NaAlO_2 + 3H_2$$

Very small amounts of the aluminum powder are necessary to reduce expansion, 0.01 per cent by weight of the cement reducing the expansion nearly 60 per cent, while 0.05 per cent reduces it to a negligible amount. It is not understood why the aluminum is so effective. The sodium aluminate is very unstable, and in the presence of water is strongly alkaline on account of hydrolysis, and further, in the presence of air, decomposes as shown

$$2NaAlO_2 + CO^2 + 3H_2O \text{———} 2Al\ (OH)_3 + Na_2CO_3.$$

If its efficiency is due to the removal of the *Na* from the reaction, then the additions of the chlorides should be more effective, but it has been shown above that they are not. Likewise when in contact with air, the Na_2CO_3 formed by the breakdown of the aluminate should cause expansion, because sodium carbonate acts very much like sodium hydroxide in these reactions. The other possibility lies in the void space formed by the expulsion of hydrogen gas which greatly fluffs up the specimens, and offers innumerable small voids to relieve any expansion. If this is true, then the mortars containing the higher percentages of vinsol resin should have shown less expansion, because in that case also there was a high volume of void space caused by air entrapped while the mortar was being mixed.

Puzzolanic or siliceous admixtures

The study of correctives has led into the field of puzzolanic materials. The value of a cement of this type was first realized when a high silica cement, with an alkali content of 0.78 per cent failed to show expansion

with a sand that developed a highly expansive reaction with standard cements of equal alkali content.

The first thought was that any apparent beneficial effects of a siliceous admixture might be due to a reaction between the alkalies and the added siliceous particles in the cement before the alkalies had any opportunity to react with the soluble silica in the coarser aggregate particles. Surprisingly, however, several series of tests using a constant weight of cement and 5, 10, 15, 20 and 30 per cent of finely powdered siliceous material did not indicate a proportional reduction in expansion, even with the 30 per cent addition.

Of the four siliceous materials tested in this series, "Monterey Shale," essentially a shale from the Miocene period, when crushed and calcined at 1000 to 1200 F., was most effective. This shale has very good puzzolanic properties when measured either by compressive strength in combination with calcium hydroxide, or by the "floc method." However, other silica dusts having equal or greater puzzolanic properties, when measured by the same tests, were apparently ineffective in reducing expansion, even when added in amounts up to 30 per cent.

A second series of tests was then put up in which the puzzolanic material was substituted for an equivalent weight of cement, the substitutions ranging from 0 to 50 per cent in increments of 10 per cent. As a measure of the effect of the siliceous material, one set was put up in which the cement was decreased, but no siliceous material added to take its place, except the added fine aggregate to compensate for the reduction in cement.

In this second series (Fig. 6), ground Ottawa sand, a crystalline silica was effective only in the higher amounts. On the other hand, the Monterey Shale, Opal and rock No. 28038 (an opaline chert) were effective in the 20 per cent substitutions, at which the expansion was negligible. When compared to the results when the cement was reduced, but with no puzzolana substitution, at least part of the reduced expansion, can be attributed to an alkali-admixture reaction.

Effect of type and amount of alkali

Some study has been made of the relative influence of sodium hydroxide and potassium hydroxide, although, as California cements are generally richer in the sodium than the potassium hydroxide, our work on the latter has necessarily been limited. However, in one series of tests, 0.89 and 1.78 per cent $NaOH$, and 1.25 and 2.50 per cent $K(OH)$ were added to a low alkali cement, the lower additions bringing the alkali content of the low alkali cement up to, and the higher additions bring the same cement up to 1.6 times the alkali content of the high alkali cement. In this series, the high alkali cement, the low alkali

cement + 0.89 per cent $Na(OH)$ and the low alkali cement + 1.25 per cent $K(OH)$, all developed approximately the same expansion, indicating that when used in equi-molar amounts, the sodium and potassium hydroxides react very much the same.

In order to study the effect of the amount of alkali as influenced by the cement factor, a series of tests were made using high and low alkali cements in 1:1, 1:2, and 1:3 mortars with alkali concentrations as follows:

CEMENT		GRAMS OF ALKALI PER GRAM OF NO. 28039 IN THE MORTAR*		
Ident.	Percent Na_2O			
		1.1	1.2	1.3
AS	0.45	0.073	0.036	0.024
GS	1.14	0.184	0.091	0.061

*5 per cent reactive rock No. 28039 added to the sand content of the mortar.

The results of this series have been rather interesting. All of the specimens made from the high alkali cement started to expand early and have shown continuous expansion. The 1:1 specimens showed the highest early expansion, but were overtaken and passed by the 1:2 specimens at 5 months. The 1:3 specimens have now, at 21 months, practically overtaken the 1:1.

None of the specimens containing the low alkali cement showed any expansion for the first six months, and up to 21 months, the 1:2 and 1:3 specimens have shown only shrinkage. However, shortly after the six months period, the 1:1 low alkali cement specimens began to expand, and at 21 months show expansion almost equal in magnitude to that of the 1:3 high alkali cement specimens.

It is not understood why the 1:1 specimens made with the high alkali cement failed to maintain the highest expansion. However, in the series mentioned above where $Na(OH)$ and $K(OH)$ were added to the cement, it was noted that the lower per cent added showed a high expansion equaling that of the high alkali cement. The higher additions (1.78 per cent $NaOH$ and 2.50 per cent KOH) did not show expansion to the same extent; rather, the expansion with the $Na(OH)$ was only .08 per cent and the $K(OH)$ only .03 per cent at 18 months. Is there an alkali content above which the reaction is retarded instead of accelerated? Possibility that such is the case is strengthened by the results described above in the series where $NaCl$ and KCl were added to the mortars. In this series, a maximum expansion was developed when the amount of the added chloride was far below the maximum addition.

By MR. NICOL*

THE NEED FOR A COMPARATIVE PETROGRAPHIC STUDY OF VARIOUS KNOWN REACTIVE AGGREGATES

Concrete failures in various areas of the country involving aggregates of many types has swelled the list of reactive materials to at least a score or more. As this list grows, there comes a great need for the study, correlation, and tabulation of the petrographic features and peculiarities of each rock to ascertain with some certainty, if possible, which mineral constituent or combination of minerals in the rock composition is responsible for the expansive reaction.

Several rock types have to date been definitely proved to be injurious. Our own California experience has emphatically demonstrated the expansive reaction tendencies of siliceous magnesian limestone, opaline chert, opaline cherty shale, opalized wood, and other materials that contain opal.† Tests on the mineral opal in practically pure form reveal this mineral definitely is the cause of expansion in many aggregates.‡

In California, vast areas of Miocene sediments contribute these opaline sedimentary materials to the streambed aggregates in percentages varying from 1 to 25 or more. It is evident that at least one of the major causes of our concrete failures is directly related to these opaline materials and necessary steps to reduce the proportionate amounts of such deleterious particles have been made.

However, we are now confronted by reports of concrete failures from various localities in which rhyolite, andesite, phyllite, felsite, granite, feldspar, and many other types of rock are believed to be a causative factor.** In some reports no mention is made as to the types of aggregates involved. Wherever possible, an attempt has been made to secure samples of these aggregates for comparative study. To date, several of these types have been secured including the andesites and rhyolites from the Parker Dam, the phyllite from Buck Hydro Plant in Virginia, the Cowlitz River sand from the State of Washington and several chert gravels from Kansas, Missouri and Illinois. Our petrographic examinations of the past mentioned chert gravels reveal that a large percentage of the chert particles contain mainly chalcedony or cryptocrystalline silica. In addition, an optically isotropic mineral having the properties of opal was detected as minute inclusions throughout the ground mass

*Mineral Technologist, California Division of Highways.

†T. E. Stanton, "Expansion of Concrete Through Reaction Between Cement and Aggregate" Proceedings of the A. S. C. E., Dec. 1940.

‡T. E. Stanton, id., Sept. 1941.

**H. S. Meissner, "Cracking in Concrete Due to Expansive Reaction Between Aggregate and High-Alkali Cement as Evidenced in Parker Dam", ACI JOURNAL, April, 1941; *Proceedings* V. 37, p. 549.

H. A. Kammer and R. W. Carlson, "Investigation of Causes of Delayed Expansion of Concrete in Buck Hydroelectric Plant", ACI JOURNAL, June, 1941; *Proceedings*, V. 37, p. 665.

Bailey Tremper, "Evidence in Washington of Deterioration of Concrete Through Reactions Between Aggregates and High Alkali Cements", ACI JOURNAL, June, 1941; *Proceedings* V. 37, p. 673.

in many of the particles. It would be of interest to analyze typical eastern and midwestern cherts further to determine the amount of opal present as the inclusion of a very small amount has caused an expansive reaction with high alkali cement in California. It is also possible that the fine aggregates from these areas may contain even higher percentages of opal than the coarse aggregates, but typical cherty sands from midwestern areas have thus far not been given sufficient detailed study to determine the frequency and concentration of opal.

A petrographic examination of the samples we have available of the Cowlitz sand from Washington, the andesites and other volcanic rocks from Parker Dam, and the Phyllite from Virginia has thus far either failed to indicate the presence of opal or at least lends support to the contention that they do not appear to contain it in any percentage which could account for their claimed reactive properties.

It at once becomes apparent that there may be some other mineral, or possibly a combination of minerals, responsible for the expansive reaction of these rock types. Inasmuch as the Parker Dam rocks and the Cowlitz River sand are largely volcanic in origin, evidence appears to point towards some mineral other than opal causing the trouble, possibly some silicate mineral having properties related to those of opal. The occult minerals in the groundmass of various volcanic aggregates, therefore, become a problem of immediate concern, and work on this phase is now underway. The phyllite from Virginia may possibly contain some form of cryptocrystalline silica that is reactive, but it does not appear that this silica is of an opaline type.

It is noteworthy that the common rock names now associated with various failures are confusing and inadequate for indicating any potential reactive tendencies the rock may have. Certainly the great percentage of andesite and rhyolite aggregates that have been used the world over have been very satisfactory in concrete. The same must hold true with felsite and granite and any of the others that now have acquired a certain stigma because of their association with some concrete failure. The *reactive mineral ingredients of these rocks must be known!* When this information is available most of these rocks will, in all probability, be found to have at least one, and possibly two or more, mineral components that are definitely responsible for the expansion. There may be several reactive minerals that thus far have escaped detection.

THE SIGNIFICANCE OF OPAL

To date, in our laboratory investigations, opal is the only *mineral* found to be reactive, at early stages at least, in combination with high alkali cement. The one notable exception to this statement is the mineral selenite (crystalline hydrous calcium sulphate, or gypsum) which causes

considerable expansion. The cause of the expansion in this case is not regarded as of the same type of reaction as in the case of opal. It is believed that the selenite causes an expansion through combination with tricalcium aluminate in the cement to form a tri-calcium sulpho-aluminate, with an attendant increase in volume.* This expansive action may not, for the present, appear to be of very great importance, but it should not be completely ignored because the presence of gypsum has been established as one of the mineral constituents in the so-called "alkali" coatings on aggregates from certain areas. More will be said about gypsum further in this article under the subject of "Reaction Rims vs. Surface Coatings."

Dismissing the selenite or gypsum for the time being and proceeding to a consideration of opal, proof that this mineral is responsible for the expansive reaction in concrete has an interesting history. The siliceous magnesian limestone No. 28039 was the first rock definitely proven to be reactive in concrete, and being available in sufficient quantity from a ledge deposit, it was adopted as the standard for all comparison tests. The rock was analyzed both chemically and petrographically. From these analyses the mineral composition of the rock was formulated. Samples of each mineral species present were then secured, and fabricated separately into expansion bars with a high alkali cement. The various mineral species were obtained either from commercial mineral companies, or from California localities. The results of these studies indicated expansion only with opal. None of the other species of minerals present in the rock gave any expansion.

The problem of isolating the reactive mineral or minerals in any rock is involved and difficult. It is suggested that a systematic method of approach can be applied to any suspected rock along lines similar to those used on the siliceous magnesian limestone.

Several varieties of opal were tested for expansion, and in all cases an expansion of considerable magnitude was observed, depending on the percentage used. Whether all varieties of opal are equally reactive is not yet known, nor has the composition of the most active opaline material been established. It is felt, however, that the water content of the opal is of great importance concerning solubility in alkalies, and that probably the opal with high water content may be more reactive in concrete than one with low water content. Further work needs to be done on this subject.

Most of the other minerals which were tested in the same manner as opal gave either negligible expansion, or else showed shrinkage. Thus, the tests on opal have to date been conclusive. The results have great

*T. E. Stanton and L. C. Meder, "Resistance of Cements to Attack by Sea Water and by Alkali Soils", ACI JOURNAL, March-April, 1938; *Proceedings* V. 34, p. 433.

significance inasmuch as opal is one of the predominating minerals in certain siliceous shales and cherts of the Miocene formaton encountered in the streambed deposits throughout California.*

Considerable research work has been done by various investigators on the properties of opal.† A great amount of this work has been directed toward the properties of opal upon dehydration.‡ Little work appears to have been done on the expansive properties of silica gel, or on the expansive tendencies of opal or opaline material when subjected to water or alkaline solution. In considering the determinations and conclusions of the work that has been done, it seems fairly reasonable to assume that expansion is a natural attendant result upon hydration of an opaline material. Much work is needed in this field to determine the magnitude of the forces involved.

Opal is formed in various ways in nature and the mode of origin and conditions of formation are varied and complex. Comparatively little is understood about opal as a product of weathering. Under the action of various agencies feldspars, as well as other common minerals in most igneous rocks, are prone to be altered into other substances, depending on exposure conditions and the composition of the original minerals. Orthoclase, for example, when acted upon by water carrying carbonic acid gas may be chemically changed into either kaolin or muscovite with silica and potassium carbonate. The important feature in regard to our problem is that the silica originally liberated by this chemical action can, under some conditions, later be dissolved by alkaline solutions and redeposited in the form of opal. This is probably true for any variety of silica regardless of its origin. Other changes may take place during weathering, the nature and significance of which have not been sufficiently studied thus far. The experiments of Spezia are of interest in this regard.** Some of the simpler reactions help us to account for the presence of opal, theoretically, at least, in practically any weathered rock, and we should expect to find much of it on the surface of the particles. In certain instances where concrete failure has been believed due to feldspar aggregates, it is safe to assume that the weathered products were probably responsible for some of the trouble.***

All aggregates known or regarded to be suspicious should be studied to verify the presence of opal. It is highly possible that most, if not all, of such materials will be found to contain either hydrous silica (Opal) or some other, as yet unknown, but related form.

*R. D. Reed, "Geology of California".
†R. B. Sosman, "Properties of Silica".
‡N. L. Taliafero, "Some Properties of Opal", American Journal Science, Vol. 229-30, 1935.
**F. W. Clarke, "Data of Geochemistry," U. S. G. S. Bull. 770, 1924.
***J. C. Pearson and G. F. Loughlin, "An Interesting Case of Dangerous Aggregate", *Concrete*, April, 1923. Homer Hadley, "Concrete in Sea Water", *Proc.* A. S. C. E., Jan., 1941.

REACTION RIMS VS. SURFACE COATINGS

The work by the Bureau of Reclamation, the Washington State Department of Highways, and other investigators, as well as by the California Division of Highways, indicates that in some cases there is a reaction between the alkali in the cement and some mineral constituent on the surface of certain aggregate particles. The photographs shown in various articles and publications indicate very clearly that there is an altered boundary surrounding many types of aggregates.

Inspection of samples of disintegrated concrete shows that in many cases there has been action upon the surface of different aggregates. A detailed inspection and classification of the various aggregate particles indicates a wide variety in petrographic types which show this reaction rim or boundary. In the case of samples of concrete from the Parker Dam, the writer has observed what appears to be rims of soft gel-like material surrounding very hard and durable particles that so far have not been placed in a suspicious group. Further study of these reaction rims under ultra-violet light has shown many different kinds of aggregates to be involved. It would appear that the reaction rim or boundary is not always associated with aggregates that are known to cause expansion in concrete. Likewise it has been found that the fluorescent rims surrounding aggregate particles are not always related to a deleterious type of material.*

Since most gravel particles weather from the surface inward, the soluble products formed by this weathering can be leached from the material, but the insoluble products remain. After a gravel particle has been in concrete for some time it is a very difficult task to determine what products of weathering coated the particle at the time of its incorporation into the concrete. Since the alkali reaction affects some aggregates from the surface inward, it is noteworthy that this reaction does not always proceed to a very great distance within the particle. This is in complete contrast to our California experience, in which we normally find the *entire particle* consumed by the reaction. In many instances inspection of disintegrated concrete shows only the reaction products, and in such cases it is impossible to determine what type of rock was originally present. Our preliminary work, therefore, was done on those aggregate particles which were only partially effected.

During February, 1941, the writer had the pleasure of visiting the Parker Dam. A side trip was made to the Bill Williams River where most of the aggregates were obtained for the construction of the Dam. A few remaining stockpiles of this material were examined, and it was noticed that many of the aggregate particles had white or cream colored

*T. E. Stanton, op. cit., September, 1941.

coatings on their surfaces. Although the coatings are mainly calcium carbonate, several specimens having coatings were found to contain some gypsum, in addition to other substances as yet unidentified. Recent findings on alkali coatings of aggregates from California indicate that in addition to calcium sulphate and carbonate, these coatings may also contain varying amounts of sodium and potassium salts. The author believes that many reaction rims or boundaries seen in concrete disintegration are due more to surface coatings than has heretofore been recognized. This is suggested as an explanation of the association of reaction rims with aggregates not regarded as deleterious. Also worthy of note is the possibility of expansive reactions when aggregates carry very high percentages of coated particles.

AUTOCLAVE TESTS

Reference has been made in an earlier report by T. E. Stanton* to some tests on feldspars and other materials in the autoclave using normal sodium hydroxide and sodium carbonate solutions respectively under high pressure and temperature.

This work was done in a check on claims from various sources questioning the durability of feldspar aggregates in concrete, particularly with regard to a possible reaction involving the alkali in cement.

In addition to the feldspars, other types of materials were tested, including three varieties of opal, several varieties of andesite (Parker Dam), siliceous magnesian limestone No. 28039, and sands from various California commercial sources including Coyote, Oro Fino, Saticoy, Marysville, and Atascadero. The feldspars included orthoclase, microcline, oligoclase, andesine, and labradorite. The specimens were ground smooth on one face to facilitate subsequent observations. The sands were carefully graded before treatment. All specimens were submerged in the test solution for six days at a pressure of approximately 300 psi., and a temperature of 430 F.

The materials which contained opal or hydrous silica were badly corroded and dissolved as a result of the treatment with both the sodium hydroxide and the sodium carbonate solutions. All five of the feldspars (which were moderately fresh specimens, obtained through Ward's Natural Science Establishment) were only partially affected, the depth of the corrosion being of insufficient intensity to warrant further comment. The andesites from the Parker Dam likewise showed little, if any, effect *other than a surface action*. Several different types of this rock were treated representing various textural phases of the material, but none seemed to suffer to any appreciable extent, compared with the opal specimens.

*T. E. Stanton, op. cit., September 1941.

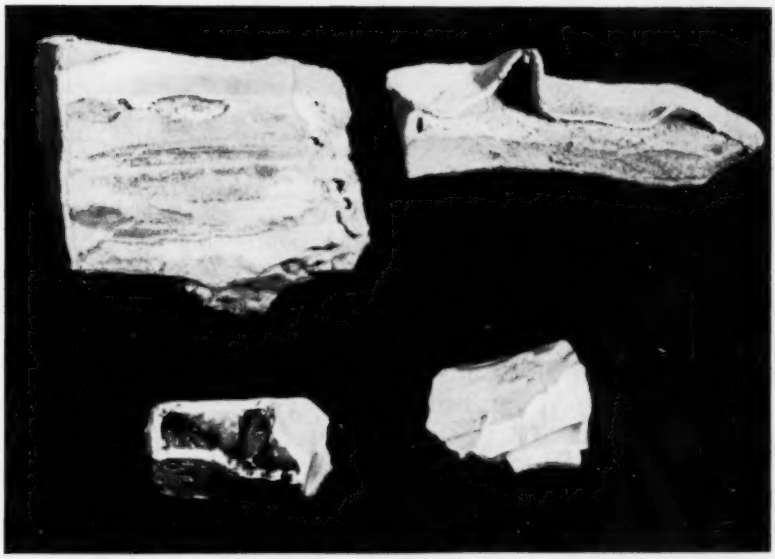

Fig. 7—Specimens attacked by NaOH solution in autoclave

Upper left and right, Siliceous Magnesian Limestone showing warping and swelling of opaline areas; Lower left, wood opal interior unaffected, with corroded surface layer; Lower right, milky vein opal also shows corroded surface layer.

A peculiar effect was noticed with the specimens of siliceous magnesian limestone, differing materially from straight opal and the other materials. This rock showed corrosive effects only in certain streaks and layers. Some increase in volume was discovered along certain portions of the specimen which resulted in a buckling and warping on the surface to a maximum depth of about ½ in., (Fig. 7). In various portions of each piece of this rock minute isometric crystals of analcime were formed. Petrographic study of these samples revealed the fact that the swelling action had taken place only in those portions of the rock which contained a high percentage of opaline silica. The three definite types of opal used in the experiment were all deeply corroded, and a porous specimen which contained finely divided opal was bleached and softened completely through by the alkaline solutions.

The sands under this very severe treatment were bleached, and the corrosive action of both alkaline solutions somewhat reduced the grain size of various minerals. It was noted that the fresh feldspars, although not originally present in a very great amount in the sands, were at least capable of surviving the test, but were etched and corroded, as were many other grains of various minerals. The oplaine chert and shale particles, however, were severely corroded and dissolved, or greatly reduced in size.

To a certain extent practically all silicate minerals including the many varieties of quartz are soluble in alkaline solutions. A consideration of the solubility of opal by comparison with these more resistant minerals shows it to be much higher in the order of relative solubility. Therefore, a test to detect opal must utilize those conditions which favor its ready or easy solubility as against the combined total solubility of the predominating amount of resistant minerals in the bulk of the sample. It was apparent at the outset that the conditions of this autoclave experiment were far too severe to expect to distinguish between the satisfactory and the deleterious sands. The test was made only to ascertain the resistance of the feldspars.

Our conclusions up to the present are that fresh feldspar certainly is not readily acted upon by alkalies under ordinary conditions of temperature and pressure. It would not appear on present evidence that feldspar should be regarded as a deleterious mineral in concrete. Other investigators have on various occasions suggested that feldspars may have been a contributing cause in certain cases of concrete disintegration. In order to link concrete failure to feldspars with high alkali cement, the solubility of the feldspar would be of paramount importance. An early reference to the solubility of feldspar in alkaline solutions is that of E. C. Stevenson.* Reference to this article indicates that the action of alkali on various feldspars is negligible under ordinary conditions of temperature and pressure, but that some action does take place with the formation of various zeolites under high temperature-pressure conditions in steel bombs for a long period.

The results of our experimental work show conclusively that the only proven reactive material encountered in the California aggregates investigated which has ready solubility in the presence of alkaline solutions is opal. Present studies may soon reveal certain silicates or occult minerals, other than opal, in the matrix of various reactive volcanic aggregates that may be a contributing cause of the high solubility of these materials in sodium hydroxide solutions†, as well as a possible cause of the failure where they were used as aggregates. Much work needs to be done on this problem, however, before any very definite conclusion can be reached.

*Journal of Geology, 1916, Vol. 24.
†R. Schwartz, Zeitschr. Anorg. Chemie., Vol. 76, 1912.

Discussion of this paper should reach the ACI Secretary in triplicate by Apr. 1, 1942, for publication in the JOURNAL for June, 1942

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN (Supplement) November, 1942

Discussion of a paper by Stanton, Porter, Meder and Nicol:

California Experience with the Expansion of Concrete Through Reaction Between Cement and Aggregate*

CONVENTION DISCUSSION: HARMON S. MEISSNER, ROGER RHOADES, LEWIS H. TUTHILL, ROY W. CARLSON, CHARLES H. SCHOLER, RAYMOND E. DAVIS, and since the Convention, J. C. WITT and closure by T. E. STANTON.

In the absence of Mr. Stanton and of his colaborators from the Convention, Professor Carlson gave a summary of the subject under discussion. Reference should be made also to other papers which serve as background: In addition to the paper by Stanton and associates published in January 1942, see also "Cracking in Concrete Due to Expansive Reaction between Aggregate and High Alkali Cement as Evidenced at Parker Dam," Meissner, ACI JOURNAL, April, 1941; "Investigation of Causes of Delayed Expansion of Concrete in Buck Hydroelectric Plant," Kammer and Carlson; "Evidence in Washington of Deterioration of Concrete through Reactions between Aggregate and High Alkali Cements," Tremper; "The Nature of the Process Leading to the Disintegration of Concrete with Special Reference to Excess Alkalis," Berkey—all three, ACI JOURNAL, June, 1941.

By H. S. MEISSNER†

In this paper is found an excellent summary of those disclosures which, within the past two years, have revealed a peculiar and pernicious action responsible for certain cases of concrete deterioration. Once the key to this particular enigma was discovered the list of mysterious failures for which it afforded explanation has multiplied. The first report may have been received with mild interest and classified as another of those agitations which have periodically disturbed the concrete industry, to wit: specific surface versus the honorable 200-mesh sieve, compound composition versus oxide ratios, sulfate resistance, autoclave expansion and special cements. But it cannot now be assumed that the malady exists in California alone, that it may be quarantined there while the

*ACI JOURNAL Jan. 1942; *Proceedings V.* 38, p. 209.
†Engineer, U. S. Bureau of Reclamation, Denver, Col.

world blithely continues to produce high-alkali cement in the hope that no uninformed engineer will combine it with any (so-called) "bad" aggregate. We must admit that much concrete is deteriorating because of reaction between aggregate and cement of inordinate alkali content, and it may be suspected that there is a great deal more. So much so, that we cannot look accusingly at the aggregates alone.

The easiest solution to this problem would appear at first glance to be simply the avoidance of reactive aggregates. Such a step would necessarily first require a fairly reliable recognition test. Bitter experience permits us to be wary of certain sources, but there does not yet seem to be any method which will, in reasonable time, allow us to prove prospective materials. However, assuming that this were possible, it is likely that a suspicious aggregate would still have to be used. For example, the fine-grained igneous rocks, such as suspected in the Parker Dam concrete, are common constituents of many natural aggregates and would not be easily avoided, much as that might be desired. Of course, if we were given a choice between two otherwise equal materials, sound judgment would dictate the use of the one found to be least active with cement. On the other hand, it is well to ponder the fact that some of the aggregates, which experience now brands unacceptable for use with high-alkali cement, have passed all acknowledged tests for judging their suitability and were considered good materials. It has been demonstrated that they will make excellent concrete with low-alkali cement and should therefore not be condemned for having been mismated. Nature has provided us with these aggregates and we should not find fault simply because man's product disagrees with them. We do not trim the foot when a shoe pinches, but secure a proper fit instead, and it seems that the same remedy is logical for the troubles under discussion.

This point of view has additional support in the growing feeling that surface crazing and cracking may be associated with alkalies in cement. Evidence that this may be true is presented in the accompanying figures. These are photographs of the top surfaces of concrete slab specimens, made with cements of various alkali contents, which have been exposed to the weather. The slabs are 4- by 24- by 24-in. and were cast out of doors and compacted for seven seconds by external vibration on the bottom of the mold after hand rodding the corners and sides. Following placing, the concrete was screeded and, when the bled water had returned into the concrete, it was finished with a minimum of steel troweling. Curing consisted of three days under wet burlap, followed by eleven days of continuous water spray, all conducted out of doors. At the conclusion of this curing period the slabs were placed on a building roof and once daily drenched with ice water.

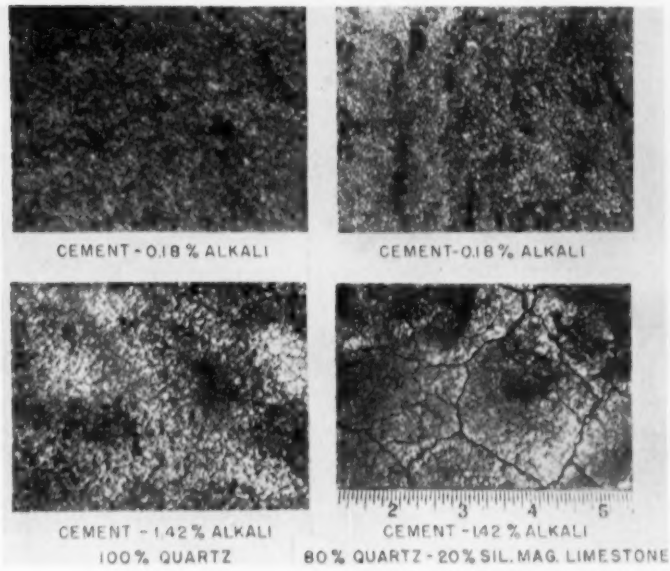

Fig. A—Concrete slabs exposed to weather
Quartz aggregate, Mix 1:2.72:2.72, W/C = 0.60, Age 56 days

The slabs of Fig. A were all composed of a crushed pegmatite quartz aggregate using four different cements whose total alkali content ranged from 0.18 per cent to 1.42 per cent. This aggregate is being used as a uniform, pure, non-reactive material for control work in studies on the action of alkalies in cement on aggregate and has to date given no indication of expansion with high-alkali cement. After fifty-six days exposure it will be noted that the amount of surface checking on these slabs is distinctly in the order of increasing alkali content of the cement used.

In Fig. B are shown two other groups of slabs in which were incorporated the two cements of greatest and least alkali content, used in the series above. The specimens on the left were made with quartz aggregate and are duplicates of the upper left-hand and lower right-hand slabs in Fig. A. Twenty per cent of siliceous magnesian limestone, which Mr. Stanton has found to be so reactive, was used to replace that quantity of quartz aggregate in making the two slabs on the right of Fig. B. The unfortunate results of combining this material with a high-alkali cement are very apparent in the lower right view. In addition to the larger expansion cracks on this slab is a finer, secondary, pattern of crazing which in varying degree is observed on some of the other slabs. However, the significant thing to be noted in this group is the absence of checking on the concrete made with low-alkali cement and quartz aggregate.

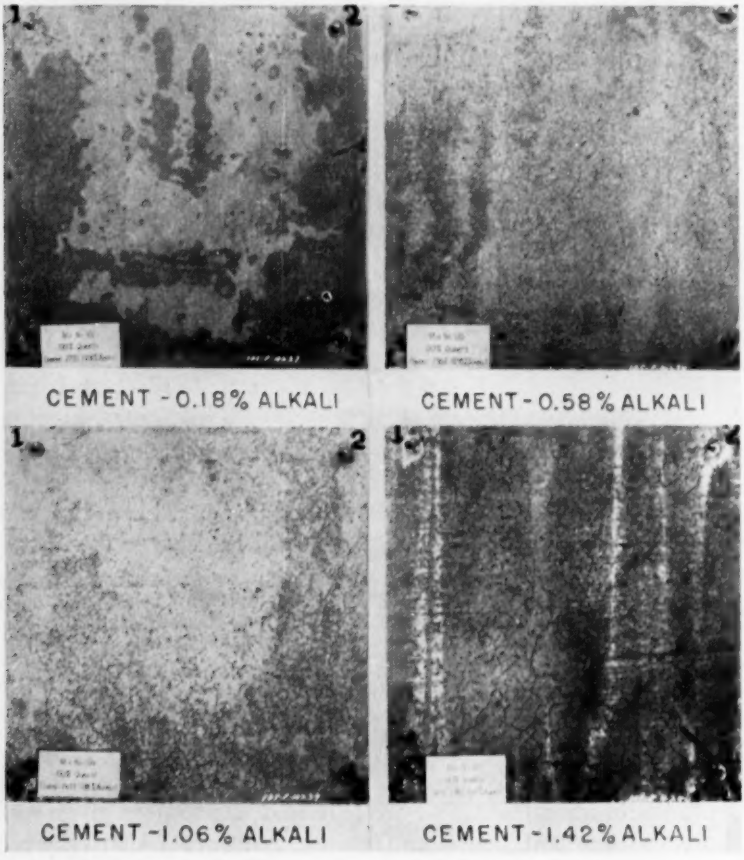

Fig. B—Close-up of concrete slabs exposed to weather
Mix 1:2.72:2.72, W/C = 0.60 by weight, Age 36 days

When this same non-reactive aggregate was used with high-alkali cement a crazed surface resulted. In fact the checking is observed more in the lower left photograph than in the upper right where low-alkali cement was combined with reactive aggregate. The results portrayed in Fig. B confirm those in Fig. A and suggest the desirability of low-alkali cement regardless of the type of aggregate used.

The reaction between aggregate and cement has been found as the explanation for deterioration in two further cases of concrete disintegration under study by the Bureau of Reclamation. One of these is a concrete pavement in Nebraska, laid in 1931 with cement found to have contained 1.35 per cent total alkalies. One year after the pavement was

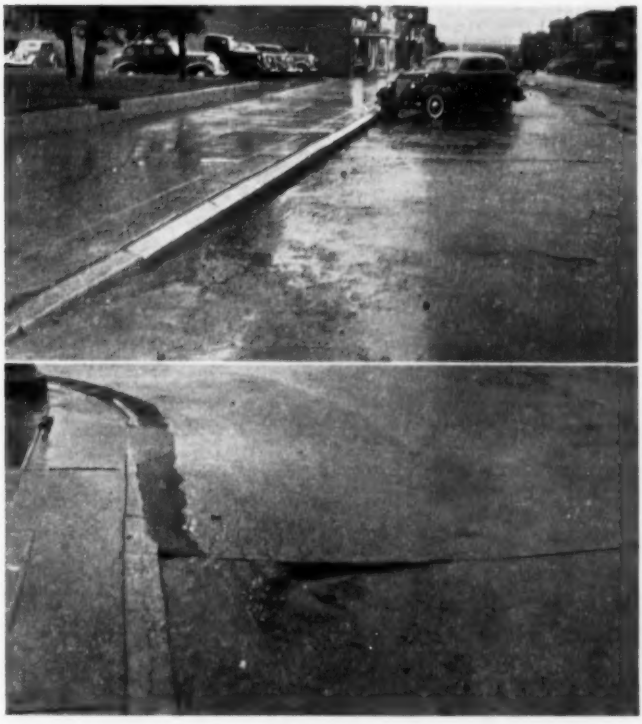

Fig. C and D—Kimball, Nebr., pavement

laid the concrete was found to be expanding to such an extent that the curbs were being pushed out of plumb (Fig. C) and the expansion joints began to show distress following their complete closure. To relieve the expansion, the joints were chipped open and a strip of sidewalk adjacent to the curb was removed. Within another few years the expansion joints were again closed, the pavement buckling and adjacent panels riding over each other. The joints were once more chipped out to widths of one to three feet and filled with asphaltic concrete which must now be periodically leveled off as the compression upon it causes it to bulge. The aggregate used in this pavement has been tested for expansion in mortar bars, using cements of both high- and low-alkali content, results of which are depicted in Fig. E. This material, which by petrographic examination was found to contain small amounts of chalcedonic and opaline silica, is observed to react with high-alkali cement but to give negative expansion with a low-alkali cement.

Mortar bar tests on another aggregate, which expands with high-alkali cement but shows no such reaction with low-alkali cements, are also

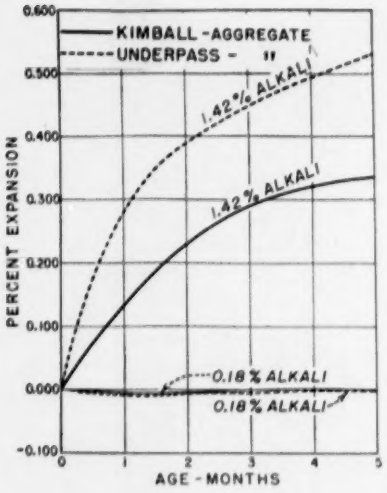

Fig. E—Expansion of 2 by 2 by 10-in. 1:2 mortar bars

Stored at 100 F. in sealed containers in presence of moisture

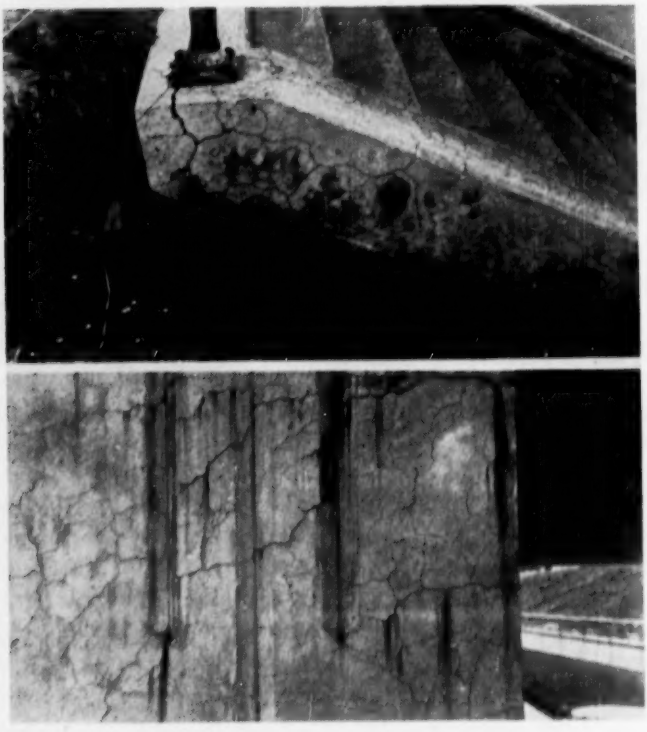

Fig. F and G—Idaho underpass, constructed 1934

illustrated in Fig. E. This material has been used in at least one important structure in Idaho, which now shows the characteristic marks of expansive cracking (Fig. F and G). Although the alkali content of the cement used in this structure is not definitely known, the present day analysis of the product from the mill which furnished the cement, and which is thought to be similar, shows it to contain 1.05 per cent total alkali.

Mr. Stanton's description of expansive cracking which has occurred in roadway concrete at Fresno, Calif., in which aggregate from the San Joaquin River was used bears elaboration, since aggregate from this same general source is being used in the construction of much important work in California. Fortunately, the Bureau of Reclamation, acting on the series of disclosures beginning with Mr. Stanton's startling article* first describing his observations on concretes and mortars made with certain aggregates and high-alkali cement, and culminating in its own experiences at Parker Dam, took immediate steps to restrict the alkali content of all cement contracted for its major construction work. On the assumption that chemically equivalent amounts of sodium oxide and potassium oxide have equal effects in concrete, a limit was established by the formula: $Na_2O + 0.658\ K_2O$ shall not exceed 0.6 per cent. By the prompt exercise of engineering judgment, when events dictated the advisability of controlling the alkalies in cement, it appears that difficulties with a potentially reactive aggregate have been avoided. The wisdom of such a move seems, in this particular case, to be established.

By ROGER RHOADES†

The studies by Mr. Stanton and his associates appear to confirm the suspicion previously held that, of all known reactive aggregates, opaline silica is perhaps the most virulent. The complications of its behaviour under various conditions of testing lead to apparent anomalies and curious ambiguities which are at present unexplainable but which will eventually prove to be invaluable clues to the specific explanation of the mechanisms involved in expansive deterioration of concrete.

The positive and conspicuous results of Mr. Stanton's experiments with opaline silica, as contrasted to the mainly negative results of his experimentation with other materials, tend to direct suspicion toward opaline silica as the cause of any given case of suspected alkali-cement-aggregate reaction. For this reason, it is perhaps pertinent to reiterate that in many instances the aggregates used in obviously affected concrete apparently

*T. E. Stanton, "Influence of Cement and Aggregate on Concrete Expansion," *Engineering News-Record*, Feb. 1, 1940.

†Geologist, U. S. Bureau of Reclamation, Denver, Col.

do not contain opal and appear to owe their reactivity to the presence of other ingredients. The papers under discussion cite Friant sand as an example of non-opaline aggregate with reactive tendencies. Parker Dam (California-Arizona)*, Buck Dam (Virginia)†, and highway structures in the State of Washington‡ are other examples, among many, which clearly exhibit expansive deterioration although apparently containing no opaline constituents. On the other hand, the concrete at Kimball, Neb., whose amazing expansion is described in a companion discussion by Mr. Meissner, was fabricated with an aggregate which contains small amounts of opal which doubtless reacted in a manner contributing to the observed deterioration.

Mr. Stanton's experiments indicate a direct production of siliceous gel by alkaline leaching of individual pieces of opaline aggregate. One is not rigorously justified in concluding, in instances in which the specific reacting constituents of a complex aggregate have not been isolated, that an associated silica gel has originated by comparable simple reaction with an unknown mineral constituent of the aggregate. If, however, one tentatively accepts that conclusion, it is profitable to examine the possible mineral sources of the siliceous gel. Opal, being notoriously unstable in alkaline solutions, is a very adequate source when present. Speculations and experimentation revolves around the possibility that under certain conditions similar release of silica might be accomplished by reaction of cement constituents with (1) natural glass, (2) certain chalcedonies, and (3) silicate minerals.

The basic volcanic glasses (amorphous, siliceous, and frequently hydrous admixtures of variable composition) have been specifically suspected of reaction in some quarters. As a common constituent of the fine-grained igneous rocks, glass commonly, if not usually occurs in western aggregates.

Typical chalcedony perhaps is no more to be suspected of potential reactivity than quartz. However, authoritative conjecture surrounds the possibility that some chalcedony verges toward an opaline character in becoming hydrous, and the question naturally arises as to the possibility of such types also assuming other similarities to opaline silica, perhaps even becoming potentially reactive in alkaline environments. Confusion surrounds this issue, partly because the chalcedonic material is frequently so fine-grained as to preclude satisfactory microscopic study; it is possible that opaline material minutely disseminated in chalcedonic aggregates may escape observation and yet impart opaline properties

*H. S. Meissner, "Cracking in Concrete Due to Expansive Reaction Between Aggregate and High-Alkali Cement as Evidenced in Parker Dam," ACI JOURNAL, April 1941; *Proceedings* V. 37, p. 549.

†A. H. Kammer and R. W. Carlson, "Investigation of Causes of Delayed Expansion of Concrete in Buck Hydroelectric Plant," ACI JOURNAL, June 1941; *Proceedings* V. 37, p. 665.

‡Bailey Tremper, "Evidence in Washington of Deterioration of Concrete Through Reactions Between Aggregates and High Alkali Cements," ACI JOURNAL, June 1941; *Proceedings* V. 37, p. 673.

which appear to pertain to the enclosing chalcedony. This point impinges directly upon the problem of the potential reactivity of cherts and gives force to Mr. Nicol's suggestion that cherts of different varieties be intensively investigated. The same considerations apply to many metamorphic rocks and to some fine-grained rocks of igneous origin which contain chalcedonic constituents.

Of the numerous silicate minerals, feldspar has come most commonly under suspicion, probably more because of its prevalence than because of any unusual chemical instability. Laboratory investigations by Mr. Stanton tend to vindicate feldspar of deleterious reactivity in concrete and the satisfactory service histories of many concretes containing feldspathic aggregate (including granite) might seem to confirm this vindication. Mortar bars containing various feldspars, made and tested by the Bureau of Reclamation have so far failed to exhibit significant expansions.

In view of our meager understanding of the mechanisms of alkali reaction, the possibility cannot be eliminated from consideration that reactions within the cement paste, with the aggregate playing no essential role or a purely passive one, may under some conditions contribute silica to form gels similar to the exudations which have been observed. Glassy phases of unhydrated clinker possess analogies to the natural glasses now under suspicion and the same analogies, although perhaps with less force, may apply to the "amorphous constituent" of hydrated cement.

Minerals other than silicates may react in concrete but not in the manner under discussion inasmuch as they cannot be sources of silica.

It can profitably be reiterated that minerals may be normally unreactive in the environment of concrete and yet rendered susceptible to reaction by weathering or by some other of the many complex and variable processes of alteration which may confuse the history of any rock, especially in igneous environments. Or, as Mr. Nicol specifically comments, the products of weathering or other alteration may be reactive and may collect in the interior or on the surface of otherwise nonreactive rocks. Simple weathering of common minerals is thought commonly to form mineral substances which could be suspected of reactivity (e.g. opal). As Mr. Nicol points out, a reaction between cement and a weathered coating surrounding an aggregate particle might form enveloping rims of reaction product which closely—although erroneously—simulated reaction with the particle itself.

Some of the earliest conceptions regarding apparent cement-aggregate reactivity in concrete involved observations of rims surrounding aggregate pebbles. Such rims are so frequently and conspicuously associated

with suspected concretes that they are commonly accepted as indicators and criteria of "abnormal" reaction, (the term "reaction rim" has come into common but unfortunately undiscriminating use). This connotation of reaction has proved to be misleading in specific instances. In all, the following types of rims have been isolated by the Bureau of Reclamation: (1) reaction rims (rims actually formed by chemical reaction between a pebble and the enclosing mortar), (2) rims of deposition (formed by the deposition around the aggregate of material derived from other parts of the concrete, (3) weathered rims (formed by weathering of the pebble prior to its inclusion in the concrete), and (4) coatings (formed by the deposition of extraneous matter on the pebble prior to its inclusion in concrete). A fifth type of rim, not yet definitely recognized by the Bureau but suggested by Mr. Nicol's discussion, would be the rim which may sometimes form around pebbles by reaction of the cement paste with coatings or weathered rims (without involving reaction with the internal substance of the pebble). It should be noted that although the term "rim" is used, exactly comparable materials may be associated with the aggregate in the form of coatings, injections along minute joints, and insinuations between an aggregate grain and its socket. Rims must be interpreted cautiously because they may indicate (1) reaction between the cement and the pebble in question, (2) reaction elsewhere in the concrete without chemical involvement of the pebble, (3) reaction with an extraneous coating or weathering product which is associated with the pebble, or (4) reaction which is in no way attributable to the cement (e. g. prior coating or weathering).

The accumulation of voluminous empirical data concerning cement-aggregate reaction, to which Mr. Stanton and his associates have contributed so compendiously, is a necessary background for the achievement of two fundamental objectives of this program of research. In the long run, those data are mainly valuable for their contribution to an explanation, in specific terms, of the intricacies underlying the expansive deterioration of concrete. In more immediate and wholly practical terms, such data must find application in tests for new aggregates and in techniques of operation which will permit the fabrication of concrete structures which are free from the effects of deleterious chemical reaction.

The tests and techniques which are so necessary for current operations are, of course, very imperfectly developed, even in empirical form; the Bureau of Reclamation, with a large number of projects in various stages of planning or construction, feels this lack very keenly. Laboratory experiments have indicated that expansive reaction can be duplicated under controlled conditions with a surprising variety of aggregate materials, but until the factors responsible for the reaction in each case are

more fully understood, it will not be possible to evaluate satisfactorily the potential reactivity of untried materials. Certain aggregates can be rejected because they contain mineral substances known to be deleterious (e. g. opaline silica). Certain other aggregates can be rejected because of their known records of reactivity in service (e. g. Bill Williams aggregate, used in Parker Dam). But in the absence of comprehensive criteria of potential reactivity, it is inevitable that many aggregates will be unjustly rejected and others, whose potential reactivity is unrecognized, accepted. Pending the development of testing techniques which will distinguish in advance between chemically suitable and unsuitable aggregates, refuge can be had only in the specification of low-alkali cement; fortunately, reactions appear to be greatly decelerated, if not eliminated in most cases, by reduction of cement alkalies.

If empirical testing techniques are regrettably lacking, we have progressed an even shorter distance toward the *explanation* of the intricacies of cement-aggregate reaction. The relation between internal chemical activity and observable external distress of affected concrete is especially obscure. These complexities cannot at present be resolved into even tentatively satisfactory explanations but, whatever the manifold causes of expansive deterioration of concrete, it is certain that the combination of certain cements with certain aggregates is one condition which will bring it about, whether, in any given case, the role of the aggregate is causative, passive, or related only indirectly. The association of expansion with certain cement-aggregate combinations is too easily observed in the laboratory to permit doubt on this point.

By LEWIS H. TUTHILL*

The authors have made a worth while contribution to the literature on dangerous reaction between certain elements in cement and certain elements in aggregate. It seems there is now no doubt but that the soda and potassa elements of the cement do bring about such a reaction, but the questionable elements of otherwise good aggregate are far less well known. The authors have ably presented information which brings this contrasting situation into clearer perspective. A few of the implications of this situation are the subject of this brief discussion.

Along with the lengthening list of deteriorated concretes in which cements having comparatively high percentages of the alkalies were used, there is a growing list of conspicuous examples, such as Mr. Stanton's Fig. 2, in which similarly exposed concretes containing aggregate from the same sources, but cements of comparatively low percentages of the alkalies, continue in service in excellent condition. This has been

*Engineer, Bureau of Reclamation, Denver, Col.

clearly demonstrated in the period of a few weeks by the contrasting condition of test slabs of concrete in which high- and low- alkali cements were used with an aggregate of proven reactivity. (Fig. A and B, Mr. Meissner's discussion.) Among the "known facts" enumerated in Mr. Stanton's paper, the importance of the alkali content of the cement is the most positive and also the most significant.

The fact that concrete of reliable durability has been made and can be made with low-alkali cement and reactive aggregates is significant because it clearly points the only way to an immediate approach to security. There is still much unknown about what constitutes reactive aggregate under different conditions, how it may be unquestionably and fully identified, and what, if anything, can be done to eliminate or neutralize the reactive elements, but it *is* known how to make low-alkali cement. A fair premium for the added costs of making it in most cases would result in only a moderate increase in price.

Because of the wide variety of aggregate components now plausibly suspected of containing elements that react with cement alkalies, at least under some conditions, it appears that the safest course, for those responsible for construction with concrete expected to give long service, is to require the use of low-alkali cement.* The premium will usually be modest or negligible when compared with the investment in the structure and the insurance against untimely disintegration. What a disappointment and a burden it is to find that a structure must be replaced long before it has been paid for, much less served its purpose.

The effects of high alkali have been so disastrous that a marked advantage has been given materials competing with concrete and the future and reputation of the concrete industry has been placed in jeopardy. In such a situation it is to the advantage of users to specify the use of low-alkali cement and to the advantage of the cement industry to encourage its use and to charge a rock-bottom premium for it. Any cement user not in position to purchase cement under his own specifications is at a particular disadvantage because he must use whatever is on the market. If that procedure leads to failure of the concrete, both the purchaser and the manufacturer of the cement are apt to be discredited. A community may be obliged to continue to pay for facilities which have long since disintegrated, whether replaced or not. Surely, when the answer is known, every one concerned is obliged to promote the manufacture and use of cement that may be used without question with any clean, structurally sound aggregate.

*Since this was written, the War Production Board has issued General Limitation Order 179 which acts to prohibit, among other things, the special manufacture of low-alkali cement, except for certain exemptions which have been or may be granted as a result of suitable appeal. Mills normally producing low-alkali cement are now its only source except by special exemption from the order.

In any event research should be continued for ultimately it will probably reveal the nature of the reactive materials and how they may be identified, the nature of the reaction in concrete and possibly how it may be prevented, and probably much else that is not as yet understood regarding concrete durability. Regardless of how much may be learned about these things, it will never be more than a fanciful hope that reactive aggregates can be processed or treated to make them safe; but we *can* make, and should have generally available a safe, low-alkali cement. No one concerned can afford hereafter to overlook this means of assuring durability.

By ROY W. CARLSON*

Mr. Stanton mentions the question of accelerated tests, and among other things he points out that the temperature ought not to be raised above 140 F. We can repeat now what we said a year ago, that the worst expansion seems to obtain when the temperature is approximately 110 F. The worst temperature is not the same for all aggregate we have tested, but about 110 F. produces greatest expansion for most materials. The time required for beginning of the expansion is a little less with a still higher temperature, but the ultimate expansion does not appear to be as great. We now use both 70 F and 110 F for most of our tests and a good many other laboratories are doing likewise.

The lower temperature is necessary to represent usual exposure and the higher temperature is helpful to get quick results because the speed can be multiplied many times by raising the temperature.

Another effect relating to acceleration of expansion is that of size of the aggregate. We find in general that the smaller the size of the aggregate, the greater and quicker the expansion, particles between 50 and 100 mesh being the worst. But in the case of extremely reactive materials, there seems to be a reversal of trend and sizes of about 30 or 50 mesh cause worse expansion than smaller sizes. A possible explanation is that the extremely fine material may react with the alkali while the concrete is still plastic and thus remove some of the potential expansion.

Regarding the effect of aluminum powder, we found also that a small amount (about .01 per cent by weight of cement) does reduce the expansion. In the case of Stanton's very reactive material, we found that a small enough percentage of aluminum to reduce the weight of the concrete only very slightly reduced the expansion materially. The explanation has not appeared as yet; the amount of aluminum is too small to react with much of the alkali.

Mr. Stanton's paper indicates that opal is the only clearcut mineral that shows the aggregate-alkali expansion. In our tests we have included

*Massachusetts Institute of Technology, Cambridge, Mass.

many pure minerals and a number of them do give excessive expansion with high alkali cement. I will mention only one, mica; five per cent of muscovite mica in the aggregate causes excessive expansion in a few months' time with high alkali cement in our accelerated test.

By C. H. SCHOLER*

This same problem or one very closely associated with it, has been observed as existing in Kansas, Nebraska, western Iowa and western Missouri for many years, probably 20 years or more. It first came to my attention about 1930 and several very brief preliminary studies were made to find out what might be the cause of the action. Because of the complexity of the problem and the rather superficial area covered in these studies, no definite conclusions have been reached.

The one striking difference between the conditions observed in this area and those reported by Mr. Stanton is that in so far as the action has been reported to the speaker it has occurred only in this area in concrete mixtures deficient in coarse aggregate. The aggregate used in this area consisted of the natural sand-gravel deposits lying in the Platt, Republican, Kansas, and Arkansas river valleys. Without exception the amount of coarse material found is not sufficient to provide for a standard concrete mixture. The actual coarse material (plus ¼-in. size) varying from 5 to 35 per cent as the material is actually used in construction. The conditions as observed can probably best be brought to your attention by quoting from a report which I made to the Kansas State Highway Department last fall. In this report I made the following comments:

1. The amount and distribution of distress and deterioration observed indicates that it is of the utmost importance that the cause and means of eliminating the trouble be found.

2. The variation in the performance record of different brands of cement would indicate the probability that the character of the cement contributes to the deterioration observed.

3. The fact that this type of disintegration in Kansas, Nebraska and Missouri is, in so far as studies to date indicate, confined to the use of so-called sand-gravel mixtures definitely shows that this aggregate is of the utmost importance in influencing this type of disintegration.

4. Reports of a similar type of disintegration in California, and by the U. S. Bureau of Reclamation have, in many instances, been attributed to free alkali (sodium and potassium) in the cement combining with certain aggregates and producing destructive volume changes. This disintegration is so similar to that observed here, that the possibility of this being the active cause cannot be overlooked. Information to date, as based upon chemical analysis of the cements concerned, does not tend to indicate this relation. The cement having the poorest record is one having a low alkali content as compared to the rest in this series.

*Kansas State College, Manhattan, Kan.

None of the cements has a high alkali content. All are relatively near each other, and of medium alkali content. It should be noted, however, that some are above the limit now recommended by the U. S. Bureau of Reclamation.

5. Preliminary studies indicate that sand-gravel from certain areas and chatt mixtures of almost the identical gradation, when used in similar concrete, seldom, if ever, develop this trouble.

6. This type of failure, while only recently given wide recognition, has been noted in structures and pavements in this area that are over 20 years of age (none included in this report).

You will note from this that I did not rule out the fact that alkali content of the cement may have been the causative factor. However, all the observed instances of this difficulty have been found only in concrete markedly deficient in coarse aggregate. The alkali content of the cements involved in this study and the service record are shown in Table A:

TABLE A—CEMENT RECORDS BY BRANDS NORTHWEST KANSAS CONDITION SURVEY

Brand	Number and Condition of Projects				Na_2O	K_2O	Eq. Na_2O
	Bad	Fair	Good	Total			
A	15	15	7	37	0.195	0.410	0.464
B	12	12	28	52	0.205	0.501	0.534
C	11	4	12	27	0.278	0.631	0.693
D	11	4	17	32			
E	10	5	11	26	0.260	0.459	0.564
F	7	6	5	18	0.211	0.377	0.459
G	7	4	9	20	0.286	0.539	0.640
H	4	23	37	64	0.232	0.394	0.491
I	1	1	5	7	0.203	0.500	0.531
J	1	6	19	26	0.199	0.317	0.408
K	1	1	—	2			
L	—	—	2	2			
M	—	—	1	1			

You will note that there seems to be no particular relation between the severity of action as observed and the alkali content. This leads me to raise two points in regard to the conclusions now tentatively accepted, that this type of disintegration can be controlled by limiting the alkali: First, granted that the reaction between the alkali of the cement and the aggregate is the sole cause of the deterioration does the limitation now imposed as to alkali content insure freedom from destructive action or does it merely postpone the time of development? Second, Is there any possibility that cement X, for example, with a somewhat higher alkali content than cement Y may have a relatively high portion of the alkali effectively bound chemically so it will not cause disintegration, and even though it does show higher alkali content than cement Y, it might be less active.

It seems to the speaker that it is likely to prove very difficult to control the unsatisfactory reactions between cement and aggregate by arbitrary controls of the composition of the cement. If this reaction is the cause of the unsatisfactory conditions observed in Kansas and the primary cause is the alkali content, it would seem that very severe restrictions might be necessary; probably both aggregates and cements and combinations of these materials will need to be thoroughly studied.

Most certainly the problem is one worthy of our best efforts and one which in the future must be considered in all major concrete construction projects.

By RAYMOND E. DAVIS*

I should regard it as unfortunate if we gathered the idea that the alkalis were solely responsible for all of the ills that we have found in concrete so far as expansion is concerned, though it has, I think, been pretty well demonstrated that in certain cases the alkalis, in combination with reactive aggregates, have been responsible in a very large measure for the excessive expansions that have been observed. One thing of interest to me is the wide variations in results of parallel tests made in different laboratories using different sands presumed to be non-reactive, with a small percentage of Stanton's reactive aggregate. I believe I am correct in saying that in the laboratory of the Bureau of Reclamation, Stanton's reactive aggregate with Platte River sand,† which was assumed to be non-reactive, exhibited practically no expansion with a high alkali cement; whereas, in the laboratory of the University of California, Stanton's aggregate in the same amount and size range added to a crushed limestone (nearly pure calcium carbonate) sand exhibited a more rapid and larger mortar bar expansion than to my knowledge has been found by any laboratory where the non-reactive aggregate was of siliceous character. It thus appears that the magnitude and rate of expansion which may be produced by a given amount and size range of reactive aggregate and a given cement may depend largely upon the remainder of the aggregate constituent. In other words, the presence of a particular mineral in small quantities in one sand may be productive of harmful effects; but in another sand the same mineral may be harmless. There are a great many angles to this problem; it is not one to which we will get the answers immediately.

By J. C. WITT‡

For the last year or two, the effect of alkali compounds on concrete has been a particularly active subject. T. E. Stanton observed some crack-

*Professor of Civil Engineering, University of California; Chairman of the Convention session.

†Professor Davis wants to make clear that he refers to Platte River sand *at Denver*—a long way from Mr. Stanton's source.

‡Technical Director, Marquette Cement Manufacturing Co.—submitted subsequent to the convention.

ing of concrete, and on the basis of a hypothesis, he made changes in the selection of materials which apparently has improved the situation greatly. There is a tendency, however, for this hypothesis to be accepted widely. This is not justified, on the basis of the evidence released so far.

After listening to papers read at meetings of technical societies, and after reading papers published in journals, I am still waiting for supporting evidence in the form of numerical data. I do not question that cracks develop in some concrete, nor that some cracked concrete contains aggregates of undesirable quality, and cement having more than 0.6 per cent of alkali compounds (calculated to sodium oxide). On the basis of the meager information that has been released, it would be a bold statement to say that the observed cracking is due to alkali compound reactions. The boldness increases when the alkali compounds are restricted to those present in the concrete as placed, and then restricted further to those present in the cement as manufactured.

That the deterioration of concrete is a serious matter is obvious. However, statements on the mechanism by which the deterioration has come about should not be accepted until evidence has been accumulated to demonstrate the proposition beyond a reasonable doubt. The acceptance of statements, the accuracy of which has not been demonstrated, could bring about some very undesirable situations. For example, it would lead, probably, to unjustified limitations for the selection of cement and aggregates, and it would tend to retard further investigation.

AUTHORS' CLOSURE

With the passage of time the full significance of the effect of the sodium and potassium constituents of portland cement assumes a broader as well as a more perplexing aspect.

The studies of the Reclamation Bureau as reported by Meissner not only check the data developed through the California studies but indicate that smaller quantities of alkali than now tolerated may be objectionable, even with apparently unreactive aggregates.

Messrs. Meissner, Rhoades and Tuthill have made an excellent contribution to the discussion of this topic. The work being done by the Bureau of Reclamation is thorough and outstanding in nature and therefore any contributions from that source deserve special consideration.

While there are undoubtedly many cases where, as Meissner states, there does not yet seem to be any method which will, in reasonable time, allow us to prove prospective materials, nevertheless it appears well

established that whenever opal is present trouble may be expected. Therefore, *all* opal bearing aggregates should be suspect, and when they must be used, low alkali cement should be specified. The presence of opal can usually be rather readily detected by a few simple determinations, such as float separation and petrographic analyses. Furthermore, all 1:2 mortar test specimens of opal bearing aggregates and high alkali cements expand excessively in three months at 130 F. in sealed containers and possibly even in 28 days and at lower temperatures (Tables B and C).

The most significant development reported by Meissner is the crazing developed with high alkali cements even in the presence of normally unreactive aggregates, thereby indicating that alkali is a basically objectionable constituent of cement and should be held to a minimum.

With reference to the reactive properties of some San Joaquin River aggregates similar to those used in the Friant Dam, built by the Bureau of Reclamation, the Bureau acted wisely in changing to a low alkali cement. The use of the silica (Pumicite) admixture with the Friant Dam low heat cement was an additional protective measure although this Pumicite has not been found as effective a corrective as other Puzzolana type silicas found in California (Table D).

Carlson's conclusion that 100 F. produces greatest expansion for most materials is of considerable interest. The California tests at elevated temperatures have been at 130 F. The results at this temperature have been very erratic, sometimes less than at 70 F. and at other times materially greater. Considerable work needs to be done on this phase but it can be stated that in general temperatures above normal accelerate the reactions and, therefore, provide one means of early determination of the probable reactive properties of a given aggregate. Certain unpublished data of the Bureau of Reclamation indicates that the ultimate magnitude of the expansions at high temperature is less than at 70 F. There is ample evidence as will be noted from the data in Tables B and C, that such is sometimes the case but that this reversal is more likely to occur with highly reactive aggreagtes than with the less reactive as far as can be judged from the limited scope of the tests to date.

Carlson states that in his tests a number of minerals other than opal are showing expansive reaction with high alkali cement. Our studies also indicate that a similar reaction may take place with an unidentified mineral or group of minerals which constitute the ground-mass of some volcanic rocks. These findings not only complicate the problem, but lend support to the early contention that the alkalis in cement may contribute to an attack on constituents other than opal.

TABLE B—REACTION CHARACTERISTICS OF TYPICAL CALIFORNIA COMMERCIAL SANDS, AND A FEW OTHER SELECTED SANDS

1:2 Mortars — Expansion in Millionths of an inch per inch

Ident. No.	Source	Sol. in NaOH*	Age at Test	Cement					
				GS (1.07 Alk.)		GS +1% NaOH		AS (0.48 Alk.)	
				70 F	130 F	70 F	130 F	70 F	130 F
28060	Fair Oaks	0.69%	28 da.	120	60	160	960	−40	−90
			3 mo.	280	230	330	1490	80	−60
			10 mo.	360	1390	640	2750	120	70
28063	Stoney Creek	0.19	28 da.	150	180	160	720	10	−80
			3 mo.	240	570	310	2100	50	−30
			10 mo.	260	1340	420	4120	30	260
28064	Bakersfield	0.22	28 da.	140	20	220	210	0	−110
			3 mo.	200	30	530	220	50	−150
			10 mo.	240	500	580	970	50	−90
28070	Marysville	0.38	28 da.	100	270	160	420	−10	−50
			3 mo.	210	210	280	1060	80	−110
			10 mo.	210	1770	390	2060	10	410
28072	San Gabriel	0.13	28 da.	150	120	210	240	−30	−40
			3 mo.	220	350	280	490	10	0
			10 mo.	200	570	280	940	0	60
28073	San Fernando	0.12	28 da.	110	100	170	220	−20	−50
			3 mo.	160	200	210	350	−10	20
			10 mo.	220	320	300	420	0	90
28074	Saticoy	2.09	28 da.	130	1460	290	2530	−10	−40
			3 mo.	240	3520	1520	3340	0	−90
			8 mo.	2740	4780	2900	4220	40	1880
28080	Truckee Riv. (Nevada)	1.87	28 da.	20	−145	60	1440	−100	−220
			3 mo.	100	300	200	3720	−120	−140
			8 mo.	230	3220	2490	4950	−130	−30
28081	Radum (Livermore)	0.16	28 da.	20	0	130	430	−60	−150
			3 mo.	60	520	110	1570	−70	−90
			8 mo.	90	960	230	2320	−50	−30
28082	Russian Riv.	0.48	28 da.	−30	−90	20	80	−120	−250
			3 mo.	−30	−20	50	1210	−160	−220
			8 mo.	60	1090	150	2260	−120	−120
28083	Felton (St. Cruz Co.)	0.10	28 da.	40	−20	130	80	−30	−130
			3 mo.	50	260	150	420	−50	−90
			8 mo.	80	420	260	690	−50	−60
28084	Coyote	0.37	28 da.	40	−120	120	180	−100	−200
			3 mo.	10	−40	120	630	−170	−190
			8 mo.	70	120	240	1250	−150	−50
28087	Sacramento Riv., Redding	0.75	28 da.	40	−40	100	100	−90	−200
			3 mo.	−30	−20	90	960	−170	−230
			8 mo.	50	210	220	1680	−170	−160
28098	Otay Riv. (San Diego Co.)	0.20	28 da.	110	10	140	180	−10	−90
			3 mo.	110	140	110	660	−20	−110
			8 mo.	130	560	210	1520	−10	−80
28102	S. J. River (at Herndon)	0.30	28 da.	70	660	100	690	−10	50
			3 mo.	110	970	190	760	−10	340
			6 mo.	190	1170	300	780	−10	630
28110	Palos Verdes (Cr. Pea Gr.)	0.14	28 da.	20	2240	240	1700	20	80
			3 mo.	400	3160	980	1990	0	430
			6 mo.	1660	3370	1410	2250	−10	1460
28115	Std. Ottawa	Trace	28 da.	0	210	60	290	−50	30
			3 mo.	50	230	150	240	−60	−20
			8 mo.	40	260	150	250	−80	40
28116	Cowlitz Riv. (Wash. State)	2.25	28 da.	20	170	90	2370	−30	−50
			3 mo.	100	660	1630	3090	−20	30
			6 mo.	770	850	3720	3640	−20	−60
28118	Slaughter H. Canyon-San Vicente Dam	0.29	28 da.	−10	80	70	210	−100	50
			3 mo.	50	150	130	320	−30	40
			6 mo.	110	210	130	340	−100	−40
28119	Kaw River (Kansas)	0.10	28 da.	30	220	70	630	−30	130
			3 mo.	90	830	150	1180	0	220
			6 mo.	120	1100	180	1390	−10	260

*Bureau of Reclamation Method—14 to 28 mesh—21 days in 10 per cent NaOH solution. Solubility in NaOH does not appear indicative except with relation to probable opal content when solubility is high.

TABLE C—REACTION CHARACTERISTICS OF BLENDS OF RUSSIAN RIVER SAND AND THE HIGHLY REACTIVE ROCKS

No. 28038 (Opaline Chert) and No. 28039 (Siliceous Magnesian Limestone) found in all local aggregates along the California coast between Monterey and Los Angeles Counties

All specimens cured in sealed containers at 70-80 F. and 130 F. 1:2 mortars. Expansion in millionths of an inch per inch.

Ident. No.	Source	Age at Test	Cement					
			GS (1.07 Alk.)		GS & 1% NaOH		AS (0.48% Alk.)	
			70 F.	130 F.	70 F.	130 F.	70 F.	130 F.
28107A	RR Sand +2½% 28038	28 da.	−20	430	110	2580	−100	−180
		3 mo.	3520	5140	1980	3000	−30	−240
		6 mo.	11560	9160	3190	3580	50	70
28107B	RR Sand +5% 28038	28 da.	−10	−180	30	1680	−110	−240
		3 mo.	1810	300	2070	3960	−70	−320
		6 mo.	5000	3660	6440	6200	−10	−60
28107C*	RR Sand +10% 28038	28 da.	80	−90	130	−170	−70	−160
		3 mo.	320	−220	1170	−270	−20	−280
		6 mo.	470	−80	4800	250	60	−120
28105A	RR Sand +10% 28039	28 da.	430	3520	1390	2370	−40	10
		3 mo.	5490	5060	2370	2420	−90	220
		6 mo.	8470	5720	2660	2440	−90	3980

*Note the corrective properties of 10 per cent of 28038. This checks all previous tests.

On the other hand it is often difficult and sometimes impossible to positively identify all of the minerals in many aggregates and the presence of opal may escape detection. In a number of instances a thorough petrographic examination has positively identified opal in aggregates which did not appear contaminated when first examined by casual methods. In view of the fact that all of the types of opal tested in the California studies have proved to be reactive a thorough search should be made for opal in all suspicious aggregates. This mineral has been positively identified in aggregates obtained from various locations in Pennsylvania and New York. This, together, with the fact that opal has been found in the chalcedonic cherts (sometimes referred to as lussatite) of the mid-West, clearly indicates that a reaction between high alkali cements and opal could account for some of the trouble in other sections of the country than in California. As Mr. Rhoades clearly points out, the distribution of opal in rocks and aggregates is widespread and, at least, cannot be restricted to local areas of Western states as contended by several.

Professor Scholer has asked some very pertinent questions. As pointed out elsewhere, the Kansas situation has certain peculiar aspects not encountered in California. Our principal studies, of course, relate to the

TABLE D—EFFECT OF VARIOUS SILICEOUS BLENDS IN REDUCING THE EXPANSIVE REACTION OF HIGH ALKALI CEMENT AND HIGHLY REACTIVE AGGREGATE COMBINATIONS

This table shows results at 18 months. Results at 8 months were shown in Fig. 6. Aggregate RR Sand + 10 per cent No. 28039 (20-80 mesh size). Cement GS (1.14 per cent alkali) 1:2 Mortar Mix. All blending material ground to minus 325 mesh and substituted for equivalent percentage of cement.

Blend Material	Expansion in 18 Months Millionths of an Inch per Inch					
	0%	10%	20%	30%	40%	50%
	Per Cent — 325 Mesh Blend					
Opal	5350	1640	160	90	....	....
Opaline Chert No. 28038	4730	1610	190	110	....	....
Calcined Monterey Shale	5540	4230	160	70	−10	−130
Pumicite	5650	4370	2630	520	190	40
Silic. Mag. Lim. No. 28039	5370	5500	5160	2450	190	150
Ottawa Sand	5200	5660	5370	4320	2990	1720
	Per Cent Blend not known					
Cement CP (Puzzol. Type)*			1250			
Cement GP " "			2820			
Cement HP " "			−60			
	Per Cent Cement (No Blend)†					
GS Cement — R.R. Sand + 10 per cent No. 28039	100	90	80	70	60	50
	5360	5100	5140	5110	4720	3660

*Cements CP, GP, and HP are three California Commercial Brands of Puzzolana or Blended Cements. HP is the only one using a blend similar to Calcined Monterey Shale or Opaline Silica.
†In this series the cement in the basic 1:2 mortar mix was reduced in the percentages shown and replaced with an equivalent per cent of the fine aggregate.

California stiuation and are sufficiently conclusive to demonstrate that we have to a large extent solved the local problem.

Even in the Kansas case, however, there is much positive evidence that though distress may have developed with low alkali cements, there is a greater expansion with definitely high alkali cement than with the low alkali (Table B). All of the projects listed by Professor Scholer were built with medium alkali cements. Had definitely high alkali cements been used, the results might have been 100 per cent bad. On the other hand, as Scholer points out, it may well be that a high alkali cement may have a relatively high portion of the alkali effectively bound chemically so it will not cause disintegration.

It may well be impracticable to control the situation in Kansas through arbitrary controls of the composition of the cement, but on the other hand, if construction economies require the use of aggregates known to be involved in trouble, it may be necessary to find some means of controlling the situation through the cement component. In such cases a truly low

alkali cement or a cement with the alkali properly bound up should ameliorate the trouble.

Professor Davis remarks on the "wide variations in results of parallel tests carried out in different laboratories using different sands presumed to be non-reactive with a small percentage of Stanton's reactive aggregate" and gives as an illustration some tests by the Bureau of Reclamation where South Platte River sand combined with 20 per cent of the California reactive rock No. 28039 (siliceous magnesian limestone) failed to show any appreciable expansion. The answer is not readily available, but it should be remembered that some Platte River sands have been found definitely reactive and to contain appreciable percentages of opal. Referring to Fig. 5 of the A.S.C.E. paper* it will be noted that 20 per cent of No. 28039 constituted the optimum amount of this rock after which the expansion fell off rapidly. Referring to Fig. 5 of the ACI paper the rapid falling off in expansion when the highly opaline rock No. 28038 exceeded three per cent should be noted. Is it not reasonable to assume that the South Platte River sand tested by the Bureau may have contained a small percentage of opal which, in combination with 20 per cent of No. 28039 resulted in a sufficiently high combined opal content as to produce little if any expansion? There can be no doubt that under certain conditions and in certain quantities, opal is a neutralizer instead of an exciter.

Mr. Witt is unregenerate in his attitude. If he is unwilling to accept the mass of data of the type produced not only by the California Division of Highways, but by the Bureau of Reclamation as conclusive in so far as the conditions observed by these respective agencies are concerned, I cannot conceive of any type of information which would be acceptable to him. He questions the accuracy of the statements made. Erratic as the results may be in some cases, nothing could be more positive in its broad aspects than the factual data shown in the tables and figures accompanying this report.

ADDITIONAL DATA

No attempt is made in this closure to discuss in detail the experience and data presented by agencies other than the California Division of Highways, except in so far as they may relate to or check the work of this department. Certain additional information is now available, however, which is believed appropriate for inclusion in the discussion on this important subject.

Recent 1:2 mortar tests using a mixture of Russian River sand and 10 per cent of the Siliceous Magnesian Limestone (No. 28039) definitely

Proceedings of the Am. Soc. of Civ. Engrs., Dec., 1940, p. 1772.

TABLE E—COMPOSITION OF SYNTHETIC ANDESITE GLASS*

	Original Unfused Mixture	Fused Glass
Silica as SiO_2	54.3	61.8
Alumina as Al_2O_3	17.4	18.8
Iron as Fe_2O_3	3.3	2.5
Lime as CaO	12.8	7.6
Magnesia as MgO	3.9	4.0
Sodium as Na_2O	6.3	3.0
Potassium as K_2O	2.0	1.2
Undetermined	0.0	1.1
	100.0	100.0

*See Table F for reaction characteristics.

check the original series, the expansions being substantially in proportion to the total alkali content of the cement (Table I) with no appreciable expansion in 12 to 24 months with any cements containing less than five-tenths of one per cent (0.50 per cent) alkali with the exception of the High Early Strength Cement BH which at 12 months has indicated more than normal reaction.

In this connection, cements GL, BH, GLA, and BAL, for which expansions at 12 months are shown in Table I, are four special California cements, selected for more complete analyses.

Cement BH, a high early strength cement, indicates greater reactions than to be anticipated on the basis of alkali content (0.49 per cent). Repeat tests may not check these results, but the following comparative tabulation of physical and chemical data relating to these special cements may offer a clue.

Cement Number	Exp. 12 Mos.	Total Alkali	Spec. Surf.	Sugar Value Clear Pt.	Free Lime	Alka-linity	Free Alk.	Water Sol. Alk.	Floc.
GL	4040	1.25	1965	2.9	0.12	5.10	6.82	0.28	0.2
BH	1310	0.49	2465	60.1	2.27	3.95	3.72	0.15	2.2
GLA	180	0.41	1915	31.7	0.35	3.14	0.95	0.04	0.1
BAL	100	0.32	1920	2.1	0.21	3.17	0.72	0.03	0.2

Note that Cement BH is relatively high in sugar value, free lime, alkalinity, free alkali, water solubility and floc as compared with cements GLA and BAL.

Many of the figures shown in Table I are the averages of a number of repeat tests on different lots of the same brand of cement. Seldom was

there any appreciable variation between successive test series of the same type cement, the high alkali cement mortars invariably showing excessive expansion and the low alkali cement mortars relatively low expansion.

On the reverse side of the picture, some unpublished tests by W. C. Hanna* show many low alkali cements catching up with higher alkali cements within a year.

*Chief Chemist, California Portland Cement Co., Colton, Calif.

Fig. H—Typical pavement failures in Nebraska

These failures are identical in type with California failures traced to reaction between cement and aggregates. The aggregates used in the Nebraska projects shown in this figure developed excessive expansion in 60 days with a high alkali cement (see text). Aggregates from Platte River at South Bend—Pictures furnished by Nebraska State Department of Roads and Irrigation

(a) (above)—Pavement 13 miles west of Lincoln, Nebraska. Reported to be typical of concrete pavement failures in Nebraska

(b) (below)—Pavement five miles north of Gretna, Nebraska. Illustrative of some of the worst conditions of the older pavements

Role of opal in the expansive reaction

As recited in the original paper, the only positively identified reactive mineral isolated in the California studies is opal. The reactions have been directly proportional to the percentage of this mineral up to some optimum amount, after which there is a rapid falling off in effect. The first highly reactive rock isolated was the siliceous magnesian limestone (No. 28039) which because it was the first reactive rock isolated and was fairly readily obtainable in quantity from narrow ledge deposits, has become the standard against which subsequent comparisons have been and are still being made. However, this siliceous magnesian limestone contains a relatively small percentage of opal ($\pm$ 10 per cent) as compared with the opaline chert No. 28038 (80 per cent +) and for that reason produces greatest reaction when present in higher percentages than the higher opal content rock. The highly opaline rock (No. 28038) produces greatest expansion when only 3 to 4 per cent is present in the fine aggregate in a 1:2 mortar, beyond which there is a rapid falling off until negligible expansion is found at 10 per cent. At the same time, excessive expansion is found in the presence of less than 0.25 per cent of this high opaline chert (Fig. 5). Latest data in the series from which Fig. 5 was plotted show 810 millionths of an inch per inch (0.08 per cent) expansion in 16 months with 0.25 per cent of the opaline chert (No. 28038) and approximately 6000 units or 0.6 per cent with 0.5 per cent of the same rock.

Meissner presents some very interesting data on the studies made by the Bureau of Reclamation in connection with an investigation of the Kimball, Nebraska, concrete failures.

Having learned of the Nebraska situation before becoming aware of the Bureau studies, and, therefore, while still unaware of the Bureau conclusion that opal in the Nebraska aggregates was probably one of the prime causes of trouble, if not the only one, we had secured through R. E. Bollen, Testing Engineer for the State Department of Roads and Irrigation a sample of Platte River aggregate from South Bend, Neb., high in feldspar. This aggregate was used in the construction of the structures shown in Fig. H.

Only a small sample was received at first, and therefore studies to date have been limited in extent.

From the sample received, all material having a specific gravity of less than 2.38 (presumably opal) was removed by flotation. On the first flotation of the uncrushed fine aggregate 0.61 per cent float was recovered and 2.05 per cent from the retained uncrushed No. 4 mesh particles. As it was apparent that there was included opal in the heavier particles (similar to the opal inclusion in the California siliceous magnesian

limestone) the retained No. 4 particles were crushed to fine aggregate size and an additional percentage of float recovered. Undoubtedly the finer particles (crushed and uncrushed) contained an additional percentage of opaline material which it was impossible to separate by the flotation process.

All float, determined to be principally opal, was crushed to fine aggregate size and then added back to the heavier fine aggregate residue in varying percentages to determine from subsequent mortar tests the probable relation of the opaline constituent to the expansion reaction.

Following are some short time tests (28 and 60 days) on various blends of the South Bend sand:

South Bend Platte River Sand	Expansion in Millionths of an Inch per Inch	
	28 Days	60 Days
1. No opal except combined	320	320
2. 0.6% added Platte Riv. Opal	400	440
3. 1.5% added Platte Riv. Opal	410	520
4. 2.4% added Platte Riv. Opal	420	620

It will be noted that the magnitude of the expansion is in direct proportion to the percentage of opal, although appreciable expansion is to be noted even with the heavier aggregate residue from which the lighter particles had been removed by flotation. The expansion with the heavier particles may be due either to contained opal, not removable by flotation, or to the feldspar. We are inclined to believe that at least part, if not all, is due rather to opal than to feldspar.

Flotation separation of opaline materials from reactive aggregates

Following is a description of the flotation procedure used in removing low specific gravity particles from the Nebraska and other aggregates:

The object was to separate by flotation and examine petrographically any minerals present in certain aggregates having a specific gravity near opal.

The aggregates used in this experiment were as follows:

Cowlitz River gravel; Cowlitz River sand; 3 types of Parker Dam andesite; 3 different samples of andesite from a San Joaquin River source; chert gravel from Elgin, Ill.; chert gravel from Missouri; chert gravel from the Kaw River, Kansas; and Platte River gravel from South Bend, Neb.

A representative sample of each aggregate was crushed to pass a No. 16 mesh sieve. It was found by experiment that for ordinary aggregates, or aggregates not containing large amount of finely textured volcanic rocks, a material passing No. 16 sieve and retained on No. 30 was best suited for the separation process. However, it was also found that for volcanic rocks containing very small phenocrysts (as was the case of the andesites used in this study) the sample had to be ground to passing No. 30 retained No. 50 in order to liberate the tiny phenocrysts. In some instances it is advisable to go to even smaller sizes.

After grinding and sizing, the sample was weighed and placed in a solution of bromoform-carbon tetrachloride of a specific gravity 2.36. The float fraction was removed, dried, weighed, and the percentage of light weight material calculated.

PER CENT OF FLOAT FRACTION OF AGGREGATES HAVING A SPECIFIC GRAVITY OF LESS THAN 2.36

Aggregate	Per Cent Less Than 2.36 Specific Gravity
1. Cowlitz River Gravel	.095%
2. Cowlitz River Sand	.32%
3. Parker Dam andesite	Insufficient to weigh
4. San Joaquin River andesite	Insufficient to weigh
5. Chert gravel, Elgin, Ill.	.52%
6. Chert gravel, Kaw River, Kansas	2.92%
7. Chert gravel, Missouri	Insufficient to weigh
8. *Platte River Gravel, South Bend, Nebraska	3.18%

*A slightly different procedure was used on this sample. It was first sieved on a No. 4 sieve and the two fractions floated at a specific gravity of 2.38. The passing No. 4 contained .61 per cent of float and the retained No. 4 2.05 per cent. The retained No. 4 that did not float was crushed to pass a No. 8 sieve and subjected to the bromoform treatment. This treatment yielded an additional 0.52 per cent float, making a total of 3.18 per cent.

Petrographic Notes on Float Fraction: 1. Cowlitz River Gravel—Float 95 per cent glassy pumice mean $n = 1.52$, 5 per cent of Hyalite Opal.

2. Cowlitz River Sand—Float 95 per cent glassy pumice mean $n = 1.52$, probably andesite in composition. 5 per cent of Hyalite opal.

3. Parker Dam andesite—Float contained a few pieces of volcanic glass, and 2 pieces of gypsum. No opal was observed.

4. San Joaquin River andesite—Float insufficient for examination.

5. Chert gravel, Kaw River, Kan.—Float contained about 35-40 per cent opal, the remainder being calcite and quartz, with some slag and coke, etc. The opal is intimately mixed with calcite, necessitating acid treatment to liberate opal.

6. Chert gravel, Elgin, Ill.—Float contained approximately 90 per cent opal, partly as organic remains and other microfossil replacement, and spherulites. The remaining material is optically isotropic. Although the total float fraction is only 0.52 per cent, it represents considerable volume, due to its very low specific gravity and porosity.

7. Platte River Gravel, South Bend, Neb.—The bulk of this float is opal. The remainder was unidentified.

Temperature as a test accelerator

Since Kammer and Carlson reported* on the accelerating effect of temperature on the expansion reactions (in the early stages at least), considerable data has been accumulated on this phase of the subject. It is obvious that if any expansive reaction can be materially accelerated by heat and this accelerated reaction is subsequently found to parallel ultimate reactions at normal laboratory temperatures of 70 to 80 F., one of the main obstacles to an early determination of the reactive properties of a given aggregate or cement will have been removed.

Kammer and Carlson claim greatest reaction at from 100 to 110 F. It has even been intimated that while temperature accelerates the reaction, the ultimate magnitude is somewhat less than under normal temperatures. Even if this is true, excessive early expansion at high

*Investigation of causes of delayed expansion of concrete in Buck Hydroelectric Plant, ACI JOURNAL June, 1941, *Proceedings* V. 38, p. 665.

temperatures is at least indicative of a combination of ingredients to be avoided.

The work of this department at higher than normal temperatures has been at 130 F. 130 F. was selected as representative of a temperature to which concrete is frequently subjected in a pavement slab whenever the atmospheric temperature exceeds 100 F. as it frequently does in California. Records of mass concrete temperatures likewise indicate that unless special cooling precautions are taken, the temperature in the interior of the mass may be expected to reach 130 F. and even higher, and under certain conditions of atmosphere and size of mass remain at this temperature for a considerable period after the concrete is placed. The fact that greater expansions may be expected at 100 to 110 F. than at 130 F. was not anticipated in the California studies and is therefore a field not as yet explored by this department.

STUDIES OF MINERALS OTHER THAN OPAL

Synthetic andesite glass

Attention was early drawn to the fact that some mineral or combination of minerals other than opal, might be responsible for the expansion reaction of such rock types as the Parker Dam andesite, the Cowlitz River volcanics and the Virginia phyllite, with high alkali cement.* Other examples of apparently non-opaline aggregates showing reactive tendencies include the sand and gravel from the San Joaquin River near Fresno.

Andesite has been under suspicion as a reactive aggregate since the Parker Dam trouble. This rock has likewise been observed in the Cowlitz gravels and is present in the aggregates from the Fresno area. The andesites from these three areas have been examined petrographically and although there are wide variations as to type, texture, etc., they all show one feature in common and that is *their glassy groundmass.* Specimens of concrete from Friant Dam, the Belmont Traffic Circle in Fresno, the Summers Creek and Courtright Creek Bridges in Washington, and the Parker Dam have been examined to observe which aggregates were reacting. Numerous thin sections were made of various particles that appeared to be participating in gel formation. It was noted that the phenocrysts of plagioclase, hypersthene, hornblende, olivine, augite, rutile, and a few other less common minerals were in most cases unaffected by contact with the cement mortar. The glassy groundmass, however, in various instances showed signs of reaction.

It was, therefore, decided to study further the glassy groundmass of various basic or intermediate volcanic rocks.

To insure sufficient uniform material for study, and for fabrication into expansion bars, it became necessary to synthesize a quantity of

*ACI JOURNAL, Jan. 1942; *Proceedings* V. 38 p. 230.

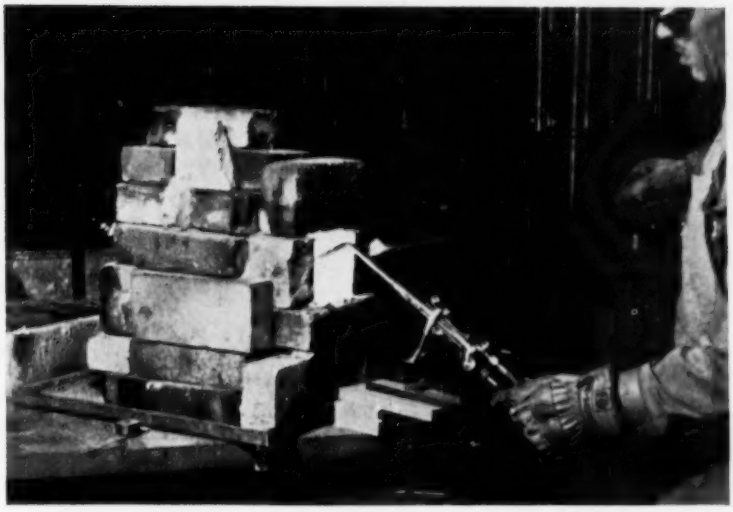

Fig. I—Furnace used to prepare synthetic andesite glass

glass having a chemical composition equivalent to that of an average andesite.* Various experiments using a carbon arc proved unsuccessful to economically prepare the quantity of glass required. By an oxy-acetylene process, however, it was possible to produce the required amount of glass in a short time.

A small furnace was constructed of fire brick. The finely pulverized charge was first moistened with sufficient water to mold it, and thus prevent the flame from blowing the powder away. The charge was made of the straight oxides of Al, Fe, Si and Mg with equivalent amounts of carbonates of Ca, Na, and K.† Chemical analyses were made of the charge before fusion, and of the final fused glass. These analyses are shown in Table E (p. 236-23).

As soon as it melted the glass was ladled from the furnace (Fig. I). After cooling at room temperature, each batch of glass was weighed, and upon completion of the entire charge, was carefully crushed, and sieved into various sizes for fabrication into expansion bars with both low and high alkali cements. Results at the end of five months based on measurement of specimens cured at 130 F. indicate this glass to be definitely reactive (Table F). The percentage of silica dissolved after 21 days in 10 per cent NaOH, using the Bureau of Reclamation method, was 0.9 per cent. Solubility in NaOH does not, however, appear to be

*U.S.G.S., Bulletin 770, p. 458.

†Compounds of the rarer elements that are usually found in andesites, such as Ti, Zr, P, Mn, etc., were omitted from the charge.

TABLE F—EXPANSION OF 1:2 MORTAR SPECIMENS OF RUSSIAN RIVER SAND AND SYNTHETIC ANDESITE GLASS (20-80 MESH) SUBSTITUTED FOR EQUIVALENT PERCENTAGES OF R. R. SAND

Expansion in Millionths of an Inch per Inch

Per Cent Synthetic Andesite Glass*	Age at Test	CEMENT			
		GS (1.07% Alkali)		AS (0.48% Alkali)	
		70 F.	130 F.	70 F.	130 F.
5%	28 days	40	90	− 50	−10
	3 mos.	110	210	0	−20
	6 mos.	130	220	− 40	−20
10%	28 days	70	210	− 60	20
	3 mos.	130	710	− 10	60
	6 mos.	80	1160	+ 10	60
20%	28 days	30	320	− 30	−10
	3 mos.	160	1630	− 10	100
	6 mos.	180	2330	− 30	240
30%	28 days	80	1150	− 10	80
	3 mos.	130	3070	− 20	420
	6 mos.	170	3980	0	1640

*See Table E for Chemical Analyses.

significant except possibly in the case of the highly soluble rocks (Table B), in which case the high solubility may be indicative of opal.

Petrographic examination of the glass indicates the mean $n = 1.541$. The range recorded is from 1.510 to 1.555. The material in the main is optically isotropic. However, occasional anisotropic grains and inclusions are observed, which may be due either to incomplete fusion, or to recrystallization. No crystallites were observed. The refractive index is somewhat higher than that of a natural glass, having a corresponding composition.*

Chemical alteration as an activator

Observations of many andesite specimens known to be reactive indicated that they were moderately or partially altered and weathered. This condition was existent prior to fabrication into concrete. Work done by Evans and Williams† in Lassen Volcanic National Park shows a few rocks in the vicinity of hot springs to be highly altered, with opal as a principal constituent.

To simulate the Lassen Park condition some of the synthetic andesite glass has been altered in the presence of SO_2 and water for comparison

*George, *Journal of Geology*, V. 32, p. 365, 1924. Grant, "Petrography and Petrology," p. 114.
†R. D. Evans and Howel Williams, *Amercian Journal of Science*, V. 29, 30, 5th Series, 1935.

of expansive reaction with the fresh glass. Various methods of alteration were tried without much success until after consulting the literature, sulphur dioxide was tried.

A portion of the untreated synthetic andesite glass, crushed to $-20+50$ size was placed in a long glass tube, moistened with water and subjected to daily evolution of SO_2 from a cylinder. A petrographic study of the SO_2, H_2O treated glass up to the present time indicates that there has been a definite change not only in the appearance of the material, but also in its composition and refractive index. The treatment will be continued for a few months longer before fabrication of the altered glass into expansion bars.

Other glasses

This work should be checked against or compared with glasses of other composition. A fertile field is indicated for the investigator who has time.

Effect of sodium hydroxide on reactive and unreactive aggregate and cement combinations

The tests for which data are shown in Table G were made after it became apparent from other tests that when the mineral under test was highly reactive without sodium hydroxide, the addition of sodium hydroxide in the amount of 1 per cent of the cement content had the effect of reducing the expansion, at least in the early stages (Tables B and C), whereas in the case of unreactive aggregate, the NaOH increased the expansion. This development led to the conclusion that there is another optimum condition which is a function of both the aggregate and sodium hydroxide, the optimum amount of NaOH being in inverse proportion to the reactivity of the aggregate. The figures in Table G appear to sustain this conclusion.

It will be noted that 100 per cent of the normally slightly reactive Russian River sand, increases in activity with increasing percentages of NaOH whereas the reactive combination of Russian River sand and $2\frac{1}{2}$ per cent of No. 28038 falls in reactivity, as the percentages of NaOH is increased.

This reverse action has been noted in other tests.

The fact that NaOH has an adverse effect on normally unreactive aggregates precludes it as a potential universal corrective.

A rapid method for detection of reactive aggregate by gel formation

Continued experimentation with samples of various reactive aggregates led to the belief that gel could be formed on a reactive aggregate if exposed to alkali solutions in a confined state similar to the environ-

TABLE G—EFFECT OF SODIUM HYDROXIDE ON UNREACTIVE AND REACTIVE FINE AGGREGATE AND HIGH ALKALI CEMENT MORTAR MIXTURES*

1:2 Mix GS Cement (1.07% Alkali)—Expansion in Millionths of an inch per inch.

Age	Per Cent NaOH added by Wt. of Cement							
	0%		1%		2%		3%	
	100% Russian River Sand							
	70 F.	130 F.	70 F.	130 F.	70 F.	130 F.	70 F.	130 F.
28 days	−10	−10	30	130	160	840	180	1060
2 mos.	20	40	70	700	240	2060	270	2330
4 mos.	−30	180	40	1440	320	2610	440	3340
	97.5% R.R. Sand + 2.5% No. 28038							
28 days	10	1340	360	2530	0	280	90	310
2 mos.	660	5800	2230	2920	60	590	140	950
4 mos.	5000	12240	3340	3630	150	850	190	1370

*This table presents figures showing that an unreactive combination may be made reactive by addition of sodium hydroxide; whereas the reverse effect may take place in the case of a highly reactive combination. This phenomenon appears to be just another illustration of the fact that one combination of ingredients may result in the development of reaction products in such a manner as to cause excessive expansion of a concrete mass and some other combination of the same ingredients may have a reverse effect.

ment encountered in the interior of a concrete mass. A series of experiments were made in an effort to produce the gel with several petrographic types of aggregate known to react with high alkali cement. A method was finally developed which proved highly successful in producing a profuse amount of gel in 5 days. This method is briefly as follows:

Each piece of aggregate is cast in a pat of neat cement. (Several pieces can usually be cast in one pat if all are of one type, otherwise it is difficult to distinguish each piece after the test.) Pats used in this work were 4½ in. in diameter, ¾ in. thick. A cement having a high alkali content and a fairly wet mix was used. After pouring into the molds, the pieces of each rock were pushed down into the mix until submerged. The pats were then cured overnight in the fog room. They were then lapped on a rotating steel lap with 220 carborundum grit until the pieces of embedded aggregate were well exposed. Next, they were allowed to remain immersed in an air-tight container (coated with paraffin) in a solution composed of N/2 NaOH, and N/2 KOH in saturated lime water for 4 days at normal temperature. Upon removal of the pats from the solution, gel was observed on or around each piece of aggregate known to be reactive. The non-reactive type did not produce any gel.

Aggregates containing opal form a profuse amount of gel by this method. In some cases, so much gel formed from highly reactive mate-

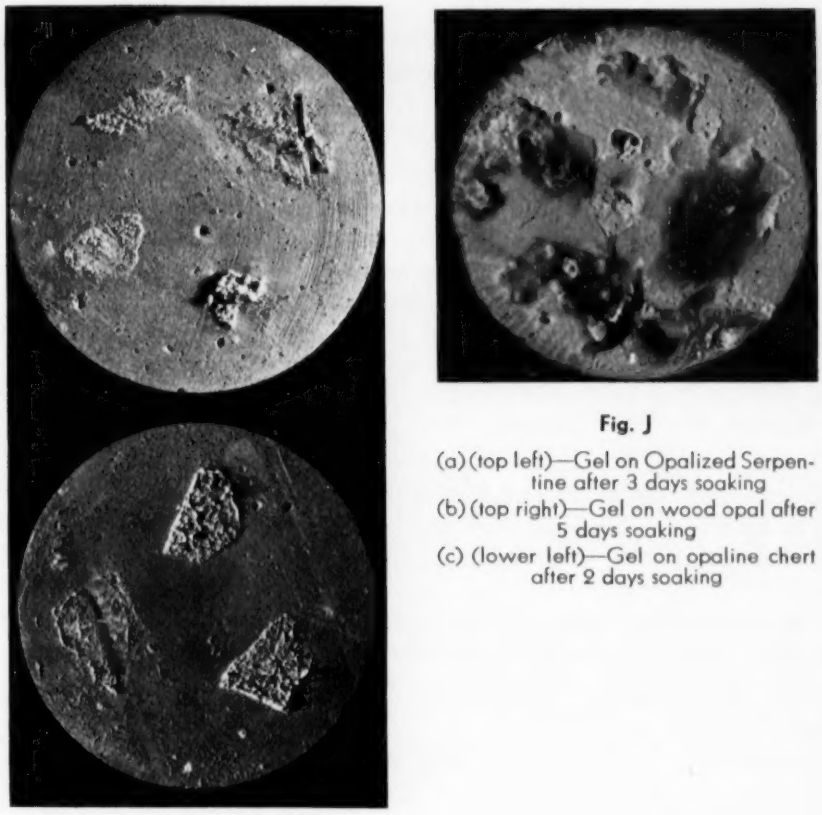

Fig. J

(a) (top left)—Gel on Opalized Serpentine after 3 days soaking
(b) (top right)—Gel on wood opal after 5 days soaking
(c) (lower left)—Gel on opaline chert after 2 days soaking

rials that the entire pat was covered. With such highly reactive material, it may be advisable to immerse for only 2 or 3 days instead of 4. This method proved fairly successful with the coarse fraction of certain fine aggregates that contain deleterious chert or other opaline materials.

Microchemical analysis of the gel formed from opaline aggregates indicates it is in all essential respects similar to gels found in concrete, although no complete analyses have yet been undertaken. It is intended to study further the action of the NaOH, KOH and $Ca(OH)_2$ solutions separately, both at normal temperature and at high temperature curing. Soaking the pat in a 1 per cent solution of methylene blue in alcohol and water to which has been added a small amount of benzoic acid is very effective in staining the gel area.*

Reaction for andesite has not been as vigorous as for opal. It is believed that a modified procedure, including curing at a higher temperature

*"Chemistry of Cement and Concrete," Lea and Desch, p. 162.

may hasten gel formation, but further work along this line has not yet been completed. It is also of interest to know whether the gel can be obtained by the same procedure as above outlined, but substituting a low alkali cement in the pats.

Siliceous blends as correctives

The hunt for correctives has continued, but except for certain ground siliceous (puzzolana type) blends, no practicable solution has been found. Studies in the silica blended cement field have been exceptionally promising in the case of the California cements and reactive aggregates. In fact, all evidence to date indicates that high silica or siliceous blended cements where the blended material is of the opaline type is a practicable solution of the problem when the expansion results from a reaction between the cement and opaline impregnated aggregate of the type encountered in California.

It will be recalled that 1:3 mortars of the blended cement HP (Fig. 4) showed no expansion in combination with the highly reactive sand (3a) up to 32 months; whereas, even the comparatively low alkali standard cements AS and BL were beginning to show definite expansion by that time.

The most recent measurements in this series at 42 months show that whereas all of the standard cements, even with the lowest alkali content, show continuing expansion, the high silica cement HP still shows some shrinkage (−170 units).

Following is the record at 42 months of all cements in this series:

Cement	Percent Alkali	Expansion at 42 Months 1:3 Mortars
GS	1.14	3380
FS	0.92	3920
EL	0.92	5720
DL	0.90	4580
CL	0.77	4460
BL	0.51	2030
AS	0.45	810
HP*	0.78	−170

*Blended

Encouraged by the results with cement HP, a series of tests was started with the highly reactive combination of high alkali cement GS (1.14 per cent alkali) and a fine aggregate containing 10 per cent of siliceous magnesian limestone No. 28039. The 8 month results in this series were shown in Fig. 6.

Sixteen to eighteen month tests are now available in this series, the results being as shown in Table D. It will be noted that silica of the

standard Ottawa sand type apparently has little corrective value, whereas the highly opaline silica (No. 28038) is very effective at 20 per cent as is the calcined Monterey shale. It is probable that only 15 per cent of this type blend would be sufficient under the most severe conditions usually encountered with commercial aggregates such as with the highly reactive commercial sand No. 1 (Saticoy).

The results shown in Table D encouraged an expansion of the series using commercial sands, cements of varying alkali content from high to low, and different siliceous blends.

The three months results of this supplemental series are shown in Table H. In an effort to accelerate the results all specimens in this series are stored in sealed containers maintained at 130 F.

Not only is this series interesting as a check on the corrective properties of siliceous blending materials from different parts of the state, but there is also an excellent correlation with results in earlier series with relation to alkali content of the cement. Note the marked drop in expansion to negligible proportions for the higher alkali cements when blended with 10 per cent to 15 per cent opaline silica.

One remarkable result of this test series is that although the California opaline silicas have definitely corrective properties, in so far as the California sands and reactive materials and the Washington State Cowlitz sand are concerned, the reverse is the case with the Kaw River, Kansas sand which is reported to have been used in concrete in Kansas which has shown distress.

Although the low alkali cements show low expansion with Kaw River sand and the High alkali cements high expansion, the addition of the minus 325 mesh reactive silica of the opaline type (No. 28038) to the high alkali cement-Kaw River sand mortars apparently has an explosive effect which increases in proportion to the increase of the opaline sliica blend up to the maximum tested (10 per cent of the cement content). At the same time, when this blending material was added to the low alkali cement-Kaw River sand combination, there was no appreciable expansion effect.

This reverse action with the Kaw River sand indicates very definitely that a corrective in one case may be an accelerator in another and somewhat complicates the problem.

A chemical analysis of the composition of the three sands, Saticoy, Cowlitz and Kaw, offers no apparent solution, although petrographic analyses indicate definite differences in mineral composition.

The Oklahoma cement (O) is reported as having been used with Kaw River sand in projects which have shown distress in Kansas. This

TABLE H—SUPPLEMENTAL TESTS

To determine potential corrective properties of siliceous (puzzolana) type blends in reactive combinations of aggregate and cement. 1:2 mortar mixtures. Specimens stored in sealed containers at 130 F. This test series supplements the earlier series for which the 18 month results are shown in Table D.

Aggregate	Cem	Per cent Alkali	Type Blending Material	% minus 325 mesh blending Mtl. Replcg. Equiv. % Cem.				
				0%	2½%	5%	10%	15%
				Exp. 3 mos. at 130 F.				
No. 1 (Saticoy)	GS	1.07%	Opal. Sil. No. 28038	2720	2740	3200	30	....
"	DL	0.86%	" " "	3510	2900	1120	120	....
"	M	0.83%	" " "	3410	2980	1230	10	....
"	EL	0.82%	" " "	3310	2630	800	100	....
"	O	0.51%	" " "	10	−30	−20	−80	....
"	AS	0.48%	" " "	0	20	40	70	....
"	IS	0.47%	" " "	−70	−100	−90	−130	....
"	JS	0.40%	" " "	−60	−80	−70	−90	....
"	GS	1.07%	" " (Berkeley)	3810	.....	3950	110	60
"	"	"	" " (Cholame)	3530	.....	3410	1160	70
"	"	"	Calc. Mon. Shale	.....	.....	4560	830	80
"	EL	0.82%	Opal. Rock Riverside Co.	2610	.....	2220	520	120
"	GS	1.07%	Vol. Tuff (Modoc)	2160	3710	3280	2010	....
"	GS	1.07A	Silica Gel	3310	3370	(1.0 %)		
RR+10% 28039	GS	1.07%	Vol. Tuff (Modoc)	7020	6950	6180	5940	
"	GS	1.07%	Opal. Sil. No. 28038	6470	.....	6040	10	
"	GS	1.07%	Calc. Mon. Shale	5710	.....	4090	70	
"	GS	1.07%	Burnt Shale No. 1	5980	.....	5310	4730	
"	GS	1.07%	Burnt Shale No. 2	6160	.....	6640	6310	
"	GS	1.07%	Silica Gel	6170	6170	(1.0 %)		
Cowlitz River*	GS	1.07%	Opal. Sil. No. 28038	1530	510	180	50	
"	DL	0.86%	" " "	500	490	230	110	
"	M	0.83%	" " "	750	140	70	50	
"	EL	0.82%	" " "	480	270	160	−60	
"	O	0.51%	" " "	−60	−70	−110	−190	
"	AS	0.48%	" " "	0	10	−40	−90	
"	IS	0.47%	" " "	60	−130	−30	−70	
"	JS	0.40%	" " "	−120	−40	−100	−130	
Kaw River†	GS	1.07%	" " "	930	1520	7210	16090	
"	DL	0.86%	" " "	1120	.....	7940	Spec.	Brk.
"	M	0.83%	" " "	900	1850	10910	20430	
"	EL	0.82%	" " "	860	1790	8260	Spec.	Brk.
"	O	0.51%	" " "	110	170	120	−20	
"	AS	0.48%	" " "	100	160	160	130	

*Sand from the Cowlitz River, State of Washington, used in concrete in that State which Bailey Tremper described in his paper published in the ACI JOURNAL, June, 1941, page 678.

†Kansas: This sand is reported to have been used in Kansas projects which have shown considerable distress. Note the reverse effect of the opaline siliceous blend to that experienced with the California and Washington reactive sands.

TABLE I—COMPARATIVE EXPANSION IN MILLIONTHS OF AN INCH PER INCH OF 1:2 MORTAR BARS

RR sand plus 10 per cent siliceous magnesian limestone (No. 28039) and cements of different alkali content. The data in this table supplements and checks that shown in Fig. 4 and Tables 2 and H

Cement*	Alkali Content Per Cent	Age	Expansion
GS	1.14%	24 Mos.	10550
FS	0.92	" "	11500
EL	0.92	" "	11800
DL-1	0.90	" "	9740
M	0.83	" "	10150
CL	0.77	" "	10450
HS	0.69	" "	12000
FS-1	0.64	" "	9640
R	0.64	" "	1150
PS-2	0.60	" "	1860
DL-2	0.61	" "	170
HA-1	0.59	" "	250
HA-2	0.51	" "	75
O	0.51	" "	660
BL	0.51	" "	50
N	0.47	" "	100
AS-1	0.45	" "	−20
IS	0.44	" "	60
CA-1	0.43	" "	−50
CLH	1.40	18 Mos.	9080
DA	0.49	" "	27
FA	0.46	" "	−30
AS-2	0.44	" "	−60
HA-3	0.39	" "	−83
CA-2	0.36	" "	−110
BA	0.25	" "	−70
GL	1.25	12 Mos.	4040
BH	0.49	" "	1360
GLA	0.41	" "	180
BAL	0.32	" "	100

*The first letter in the cement identification column indicates the source; all cements with the same initial letter being from the same source or brand. The second and third letters indicate variations in type of the same brand. The numbers following some of the letters indicate repeats of the same type cement with some variation in alkali content.

is puzzling in view of the fact that some California tests of this cement-aggregate combination fail to show any appreciable expansion in three months at 130 F., whereas all the higher alkali cements show high expansion. The results have been erratic however, as a few long time tests (28 months) at normal temperatures with cement (O) and Kansas aggregate do show as much expansion as the expansion found with the same (Kaw River) aggregate and the California High Alkali cement GS.

These confusing results to a certain extent check Professor Scholer's observations and definitely indicate that the Kansas problem has com-

plications not encountered in California where the results of all tests extending over a period of four years have been remarkably consistent.

Tests with the Kansas materials, however, do not in any sense discredit the application of the California determination of the reactive effect of alkali and opal combinations even under Kansas conditions as evidenced by the figures shown in Table H.

They do, however, emphasize the fact that other at present undetermined factors are contributing very definitely to the magnitude of the expansions.

As previously stated, two important factors may be the method of occurrence of the alkali in the cement and the reactive mineral or minerals in the aggregates.

CLOSING SUMMARY

In closing, it may be well to summarize some of the principal data presented in this discussion.

1. In so far as California experience is concerned, the data conclusively demonstrate that excessive expansion may occur in a concrete mass due to reaction between certain ingredients in the aggregate and cement.

2. One combination of ingredients producing this reaction is opal in the aggregate and the alkalies in the cement.

3. There is a certain optimum percentage of opal in the aggregate which produces the greatest expansion. This "optimum" percentage is apparently a constant with regard to *total* opal content in the combined aggregate regardless of the variation in percentage of opal inclusions in individual particles. For example, opaline chert No. 28038 contains about 80 per cent or better of opal, whereas, siliceous magnesian limestone No. 28039 contains less than 10 per cent opal. The "optimum" amount of "opal", as above referred to, is approximately 3 per cent and when present in excess of 10 per cent of the total aggregate, the reactions are of such a nature as to result in no measurable expansion under the test procedure followed.

4. In general, the expansion is proportional to the total alkali content of the cement with sufficient evidence to indicate that the results may be affected to some extent by the nature in which the alkali occurs in the cement.

5. Expansions are relatively rapid when the alkali exceeds 0.60 per cent and relatively slow, if at all, when less than 0.50 per cent. The zone between 0.5 per cent and 0.6 per cent requires further exploration.

6. The ultimate magnitude of the expansion in the case of low alkali cement-reactive aggregate combinations, as indicated by the 4 year test data, is not sufficient to cause any concern under similar conditions

of aggregate and cement combinations. This checks field experience. No trouble has been noted over a period of five to fifteen years with the same reactive aggregates used with normally low alkali cements.

7. In addition to opal, the glass matrix of volcanic rocks may be reactive. Synthetic glass of andesitic composition has definitely been proven reactive.

8. Reactive aggregate particles may be detected by means of gel formation following certain described accelerated test procedure.

9. Temperature in some optimum amount yet to be determined is usually an accelerator in the initial stages of the reaction, but may serve as a retardant in so far as ultimate magnitude is concerned.

10. Sodium hydroxide may be an accelerator or retardant depending on other variables such as amount of reactive mineral in the aggregate or alkali in the cement or a combination of both.

11. Certain siliceous (puzzolana) blends definitely operate as a corrective when the offending mineral is of the opaline chert type encountered in California, and of the reactive types found in the Cowlitz River, Washington aggregates, but may have the reverse effect with reactive minerals of the nature found in the Kaw River Kansas aggregates.

12. Some volcanic tuff and burnt shale blends are apparently accelerators, or at best have minor beneficial effects under the conditions and in the quantities tested in the California series.

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Observations on the Durability of Dry Tamped Silo Staves*

By C. A. HUGHES†
Member American Concrete Institute
and KENNETH A. ANDERSON‡

SYNOPSIS

From data obtained by subjecting cubes cut from dry-tamped silo staves to durability cycles consisting of frost action alone or a combination of frost and acid action, it is concluded that the transverse strength and absorption are not adequate criteria of the durability of dry-tamped silo staves. From a discussion of exposure conditions and evidence obtained from field inspection, it is concluded that frost action is the chief factor in silo durability though acid action is still important because of its accelerating effect on the rate of disintegration in freezing and thawing tests. The procedure for a durability acceptance test is proposed.

INTRODUCTION

The present proposed specification[1] for dry-tamped silo staves requires a minimum transverse strength of 690 psi. (120 lb. per in. of width for a thickness of 2½ in.) and a maximum absorption of 6 per cent by weight at 24 hours. The strength limits were raised from those of the preceding specification with the intention of increasing stave durability, i.e. the resistance to disintegrating factors encountered in service.

This paper is a progress report on the investigation of the adequacy of short time tests as durability criteria and the development of a durability acceptance test to be used as a substitute for, or in addition to, the requirements of the present specification. Detailed descriptions of procedure and many of the data have been eliminated. Such details of the investigation may be obtained from a Bulletin to be published by

*Received by the Institute Nov. 3, 1941.
†Associate Professor, Structural Engineering, University of Minnesota.
‡Research assistant, University of Minnesota.
(1) Number subscripts refer to references given at the end of this paper.

the Engineering Experiment Station, University of Minnesota, "Durability Tests For Dry-Tamped Silo Staves."

The investigation has been predicated on the belief that the essential requirements of an acceptance test are:

1. The test is a true measure of the quality under examination,
2. The range and distribution of the data permit suitable grading of the materials subjected to test,
3. Data can be obtained in a reasonably short time,
4. The procedure is simple and the apparatus available in most laboratories,
5. Correlation between data obtained by different operators and laboratories on similar specimens is high, and
6. The cost of the test is not prohibitive.

Because of the time element involved in durability tests and service records, it is doubtful if there ever will be general agreement that any acceptance test for durability satisfies the first requirement.

In devising durability tests, it is generally agreed that the laboratory exposure conditions should similate the exposure conditions of service. Because of requirement 3 the laboratory test must always differ from service exposure in the duration of the exposure period and, because of requirement 5, in the size and shape of specimen. Moreover the large number of variables involved in service exposure cannot be duplicated in the laboratory if requirement 4 as to simplicity is to be adhered to. Consequently the similarity between field exposure and laboratory exposure can never be very marked. As a practical matter, the laboratory test must be restricted to the inclusion of one or two of the major factors encountered in service and to small, easily handled specimens.

There is no disagreement that the authors are aware with the statement that the chief factors affecting the durability of silo staves are (1) chemical interaction between silage materials and the concrete, and (2) frost action. The first factor is commonly referred to as "acid" action and it will be so designated in this paper.

Because of the lack of adequate data, particularly field data, there is disagreement as to the relative importance of the two factors. The silo manufacturers rarely mention frost action but are quite voluble in regard to acid action. As a result of this attitude, shared by the majority of those in contact with the industry, papers on the durability of silo staves have been concerned chiefly or wholly with the corrosive effect of various liquids encountered in a silo. [2] [3]

Cubes, approximately $2\frac{1}{2}$ in. on a side, cut from dry-tamped silo staves, were subjected to three exposure cycles consisting of (1) alternate freezing and submersion in a $0.16N$ aqueous solution of lactic and acetic acid, the A-cycle, (2) alternate freezing in air and submersion in water,

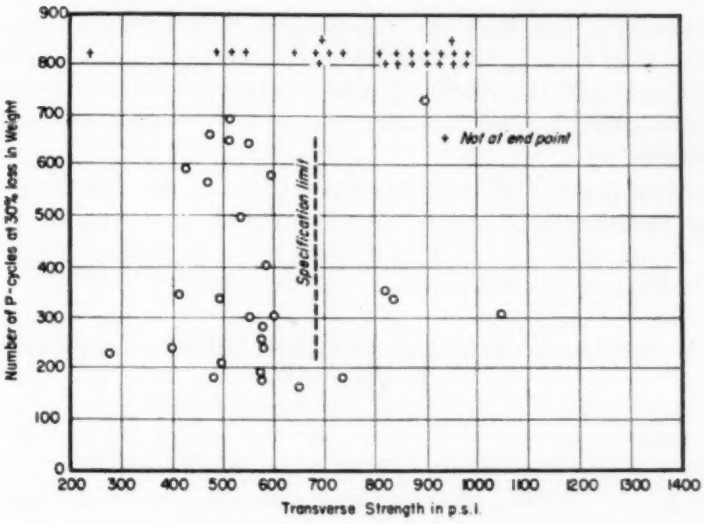

Fig. 1—Data from the P-cycle versus transverse strengths

the P-cycle, and (3) alternate freezing in air and submersion in water under pressure, the H-cycle. The number of cycles at a 30 per cent loss in weight was used as the criterion of resistance.

Two variations of the A-cycle were used. In one the cubes were frozen in pans containing 1 in. of water[4], and in the other the cubes were frozen in air. No significant difference was noted between the data from these two variations. Comparison of the data from the A-cycle and data from acid tests revealed that the A-cycle measured acid resistance only except for specimens of very low resistance to frost action. In another investigation no correlation was found between the P-cycle and natural weathering, nor was any relation found in this investigation to establish confidence in the usefulness of data from the H-cycle. Consequently, no data for either the A-cycle or the P-cycle are shown.

In Fig. 1, data from the P-cycle are plotted against the transverse strengths. Data for those specimens which have not reached an end point are shown by crosses on or near the 825 cycle line which was the number of cycles at the time Fig. 1 was drawn. The plotted points of this and of subsequent figures represent data for individual tests.

From Fig. 1 it is evident at once that no relation between the transverse strength and the resistance to the P-cycle is shown by the data for specimens which have reached their end points. It is also evident that when data for specimens not now at an end point are included that the average resistance of all specimens complying with the strength specification will be much hgher than that of specimens having trans-

verse strengths less than the specification limit. Hence, though the chance of high resistance to freezing and thawing is greater for specimens of higher strength, the absence of a relationship clearly indicates that the transverse strength is not an adequate criterion of resistance to freezing and thawing.

The inadequacy of the transverse strength as a criterion of the resistance to freezing and thawing is emphasized by data for specimens cut from cast staves as may be seen by comparing the data of Table 1 with those of Fig. 1 or merely by looking at the table.

TABLE 1—RESISTANCE OF CERTAIN SPECIMENS CUT FROM WET CAST STAVES TO THE P-CYCLE

Group	A			B		C		D		
Transverse Strength, psi.....	525	460	440	762	633	422	381	1130	1160	1189
Number of P-Cycles of 30 per C..	29	19	33	62	47	120	149	100	77	115
Loss in Weight.....								298*	188*	298*

*Specimens started in freezing cycles 3 months later than first set. The first set were about 1 month old when freezing and thawing started.

The surprisingly high resistance of the dry-tamped mortars to the P-cycle is attributed to their high air voids and the low water-cement ratios of the paste, these being properties which result from the method of manufacture. Durability studies on brick have shown that the effect of freezing and thawing is greatly reduced if the ratio of absorbed water to pore space is 0.8 or less. When the freezing procedure is such as to promote filling of the pores as in the H-cycle, the resistance to frost action is low. Investigations of treated cements have shown that increase in air-voids has accompanied an increase in resistance to freezing and thawing.[5]

Other durability studies of this laboratory have shown also that the compressive strength is not a reliable criterion of the resistance of mortars to natural weathering or to a slow cycle of freezing and thawing such as the P-cycle.[5] An outstanding example of a low strength and highly absorptive material having exceptionally good resistance to freezing and thawing is given by the performance of a cement mortar made with vermiculite aggregate. Despite a compressive strength of but 250 psi. and an absorption by weight of 70 per cent (about 44 per cent by volume), the average number of P-cycles at 15 per cent loss in weight was 420 for eight specimens, which is more than twice as many as were withstood by mortar specimens tested at the same time and having an

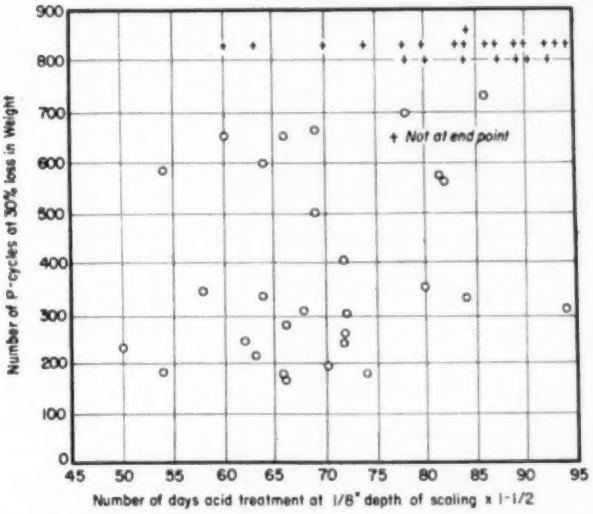

Fig. 2—Data from the P-cycle versus resistance to acid action

average compressive strength of 7260 psi. The authors know that very dense, rich, and strong mortars made with durable aggregate are highly resistant to freezing and thawing.

Despite the lack of relation between freezing and thawing resistance and the transverse strength, it should be emphasized that the chance for high resistance to freezing and thawing is greater for staves having high transverse strength. Therefore, until some durability acceptance test is devised, the strength limitation of the present specification serves a very useful purpose in that it eliminates the use of the majority of staves having low frost resistance. Compliance with the strength of specification does not, however, assure durability.

Fig. 2 is shown because there is some tendency in the silo industry to regard resistance to acid action as also a measure of resistance to frost action. It will be seen that no relation between acid resistance and frost resistance was obtained for those specimens which had reached an end point. Nor are the authors aware of any reason why there should be a relation.

In Fig. 3 the data from the P-cycle are plotted against data from three measures of absorption. It will be seen from Section B of Fig. 3 that the 24-hour absorption has no value as a criterion of resistance to freezing and thawing[7]. The same conclusion was reached in reference to resistance to acid action[5]. The specification limit for 24 hour absorption therefore appears to be unnecessary. No relations are exhibited by the data for either the five minute absorption or for the ratio of five minutes

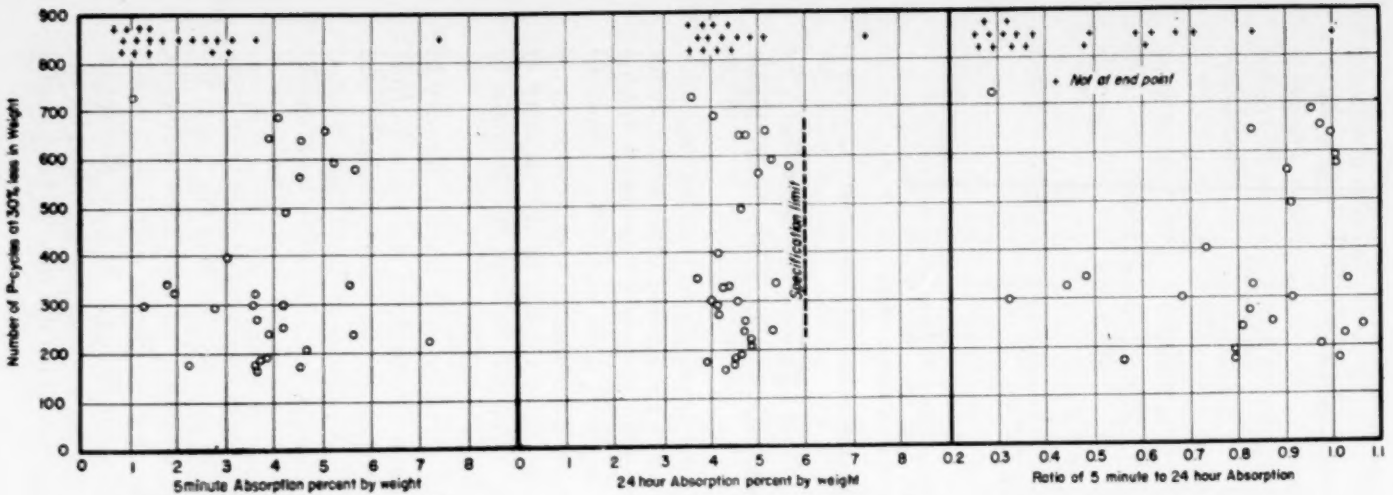
Number of P-cycles of 30% loss in Weight
5minute Absorption percent by weight
24 hour Absorption percent by weight
Ratio of 5 minute to 24 hour Absorption
Specification limit
+ Not at end point

Fig. 3—Data from the P-cycle versus three measures of absorption

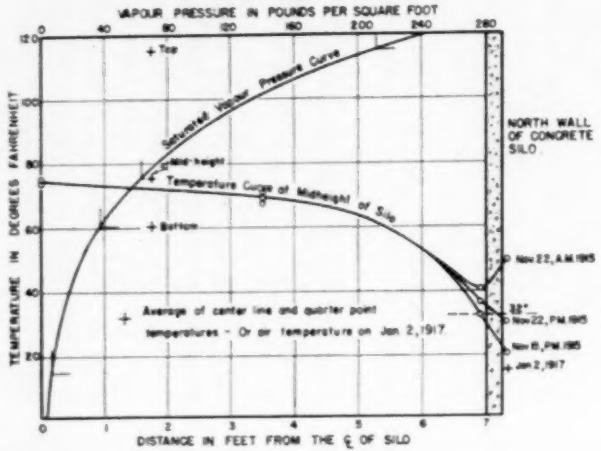
VAPOUR PRESSURE IN POUNDS PER SQUARE FOOT
TEMPERATURE IN DEGREES FAHRENHEIT
DISTANCE IN FEET FROM THE ℄ OF SILO
Saturated Vapour Pressure Curve
Temperature Curve at Midnight of Silo
+ Top
+ Bottom
NORTH WALL OF CONCRETE SILO
Average of center line and quarter point temperatures - Or air temperature on Jan. 2, 1917.
Nov 22, A.M. 1915
Nov 22, P.M. 1915
Nov 15, P.M. 1915
Jan 2, 1917

Fig. 4—Temperatures and vapor pressures in a silo

to 24 hour absorption, though it is true that the chance for high resistance to frost action is greater for specimens having low values of either of these measures of absorption.

Durability requirements in a specification are intended to ensure a minimum life in service. Until an acceptance test has been in use for two or three times the minimum life of the structure, it will not be known definitely whether or not it meets the first requirement of an acceptance test, namely, that the test is a true measure of the quality under examination. The choice, then, is between waiting twenty, thirty, or forty years, by which time conditions and materials will probably have changed radically, or accepting some test recognized as imperfect but which reason and available data indicate to have the greatest chance of satisfying requirement (a) and which at the same time meets or approaches the remaining requirements of an acceptance test. Acceptance limits for such a test should initially be set quite low and later raised if experience shows the method to have merit and a modification of the specification requirements to be desirable.

Before proposing an acceptance test for durability, or before passing judgment on a suggested method of test, service requirements should be examined as carefully as possible. Though as was mentioned previously, the silage industry as a whole regards acid action as the chief disintegrating agent, it is the authors' opinion that acid action has been highly over-emphasized and that the chief disintegrating agent is frost action. If this is so, then test procedures for durability and methods for promoting silo durability will be fundamentally different from those resulting from the acceptance of acid action as the chief factor affecting the life of a silo.

The general acceptance over many years by manufacturers, technical men, and users, of the theory that acid action is the primary cause of silo failures is due apparently to the following facts:

1. Silage materials do react with concrete or mortar.
2. The role of acetic and lactic acids in preserving silage has been widely publicized, which is true also of the fact that acid will attack concrete.
3. Whenever silage juice runs over concrete the action is rapid and leaves a prominent and permanent record of its destructive power.
4. The typical surface texture of the interior walls of silos after years of service is not similar to that usually associated with frost action.
5. Psychological reactions from the comforting thought that the presence of corrosive materials in silage and their interaction with concrete are factors beyond the control of the manufacturer.

In Fig. 4 are shown data on temperatures in a silo read from Fig. 5 and 9 of Bulletin 303, Iowa State College[8]. Temperatures were given

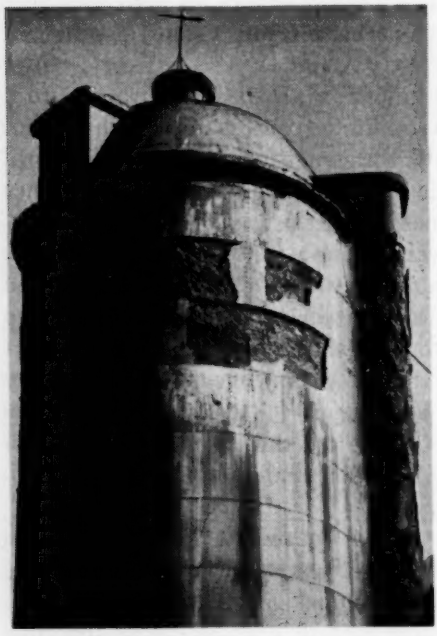

Fig. 5—Frost disintegration due to high humidity, high capillarity low strength concrete, and an exterior vapor barrier

only at the center, quarter-point, and near the wall, and consequently the true shapes of the temperature curves may vary somewhat from those shown. The relation between the vapor pressure and the temperature for 100 per cent relative humidity is also given on the figure.

Because of the high moisture content of the silage, about 200 per cent on a dry weight basis, it is reasonable to assume 100 per cent relative humidity within the silage. Above the silage level the relative humidity will be high relative to the outside air on clear days but may be less than 100 per cent. If the barn is ventilated through the silo, the relative humidity will be high and probably 100 per cent.

The vapor pressure differentials are such as to cause an outward movement of water vapor. Assuming 100 per cent relative humidity, condensation will occur as the water vapor travels to regions of lower temperature, and the rate of condensation will be proportional to the temperature gradient. Because silage temperature at the middle of the silo does not fall below about 60 F. even in the severest weather, there is always a vapor pressure differential between the interior of the silage and the outside air.

Let us consider the moisture conditions at the wall of a silo built of 2½ in. dry-tamped staves treated on the interior with a cement plaster or cement wash. Either treatment is quite effective as a water barrier.

The permeability of the stave as determined by the usual laboratory procedure may be high or low, but even if low, so long as it is a dry-tamped stave, its resistance to the movement of water-vapor, or of water under pressure, will be less than that of the coating. Assuming a temperature curve similar to that marked Nov. 22, p.m. on Fig. 4, an accumulation of moisture will occur on the interior face. Because of the resistance to moisture movement through the coating and the relative freedom of movement in the stave, it is to be expected that a wet stave will lose moisture in clear weather until an equilibrium condition is reached. Hence, in the absence of rain the moisture content of the walls will vary from saturation at the interior face of the coating to less than saturation down to dryness in the stave itself.

Concrete staves are thin in section, low in specific heat, and low in resistance to heat transfer. These properties are such as to promote close correlation between the temperatures at the exterior and interior surfaces.

Freezing at the interior face will occur whenever the air temperature remains some degrees below freezing for a sufficient time to over come the time lag. Just what outside wall temperature maintained for how long is necessary to cause freezing at the interior face is not known and will, of course, vary considerably depending on a number of factors. However, it is probable that both the temperature drop and its duration will be small, perhaps 5° for 1 to 2 hours. Because the interior face is saturated with moisture and the staves are not, damage from freezing will be relatively severe at the inner face and slight to negligible in the stave. Examination of weather records indicates the possible number of cycles of freezing and thawing is somewhere between 50 and 100 due to air temperature changes alone in the vicinity of Minneapolis.

If radiant heat is taken into account the possible number of cycles of freezing and thawing is greatly increased for the conditions assumed, i.e., a more or less dry stave and a saturated interior face. Assume a day with a sky clear except for slowly moving cumulus clouds and an air temperature of say 25 F. When the sun is shining against the wall, radiant heat will cause an increase in temperature and, if the temperature rise is great enough, thawing will occur at the inner face. If the wall is shaded, the wall temperature will drop toward the air temperature. Thus, it is possible that the passage of the shade from a large cloud over the silo face will result in a cycle of freezing and thawing. On clear dry nights, it is possible to have freezing even though the air-temperature may be slightly above 32 F.

That thawing against the wall does occur during the day time even though the air temperatures are below freezing is well known to users of silos and is employed as a salespoint by some stave silo manufacturers.

Because the amount of radiant heat increases with the angle of incidence of the sun's rays and the altitude of the sun is low throughout its orbit in northern zones during the winter months, the radiant heat will be greater on vertical surfaces than might be expected from experience with its intensity on horizontal surfaces. While the subject of maximum radiant heat has not been investigated by the authors it seems likely that peaks would occur in the midmorning and in the midafternoon. If this is so, the southeast and southwest faces of the silos, unless shaded, should have the greater number of cycles of freezing and thawing. Attempts to check this reasoning by observation in the field have not been very successful though it is reasonably safe to state that the condition of silos showing disintegration is better on the north side or on a shaded side than for the remainder of the silo.

In a report[9] of an extensive silo survey made by W. L. Kaiser and A. J. R. Curtis for the Portland Cement Association following a discussion of the condition of other types of silos, it is stated, "Block silos, however, were found to be in first class condition and in only one case was there more than an incidental etching of the interior surface." This superior durability performance of block silos has been true for the few block silos inspected by the senior author. Accepting frost action as the major factor, an explanation is immediately available in that the number of cycles of freezing and thawing at the inner wall face will be many fewer than in a stave silo because of the lesser sensitivity of the inner wall faces to temperature changes on the outer wall faces due to the better insulating properties and the higher heat capacities of blocks 6 to 8 in. thick and usually with air spaces, as compared with solid staves 2½ in. thick. In contrast, acceptance of acid action as a major factor would result in the expectation that the damage to the interior face would be increased by increase in wall insulation because this would result in higher average temperature at the wall and therefore greater chemical activity.

Laboratory tests and field experience have shown that freezing and thawing of saturated mortar or concrete will result in disintegration sooner or later depending on the quality of the material and the severity of the exposure whereas concrete more or less dry and exposed only to atmospheric temperature changes does live up to the well known slogan "concrete for permanence". Hence our assumed wall construction having a water barrier applied to the interior surface of an easily breathing stave will suffer damage to the surface only. There will be no decrease in structural stability until disintegration reaches the staves. When this stage is reached, the obvious action is repair of the surface. If this is done when required, there seems to be but one cause, to be dis-

cussed later, why the assumed wall construction may not last as long as there is use for the silo.

If the treatment on the interior or the character of the wall is such that saturation occurs within the wall then frost action will eventually cause damage which will decrease the structural stability to a degree which will require replacement of the silo.

Frost action at the interior wall faces will be greater on the areas in contact with the silage because saturation of the wall surface by condensed water vapour plus water in the contact silage is assured whenever the temperature at the wall surface falls through the freezing point. In one silo visited frequently during one winter, ice-sheets were found to have formed against the walls. The appearance of the interior of a silo is not unlike that of stucco in areas subject to drainage of water down the wall. In this case also, ice-sheets form during freezing weather. Because silos are usually emptied gradually throughout the winter the frost action will be a minimum at the initial level of the silage and increase toward the bottom. Hence frost action can be used as conveniently as acid action to explain the fact that surface disintegration is usually more severe in the lower parts of the silo. If frost action alone is considered, it follows that the condition of the walls should improve below the ground level. Such improvement has been observed in a number of silos.

The weakness of the assumed wall construction mentioned above when discussing surface attack lies in excessive porosity of the wall. Highly porous walls take in rain more or less like a sponge and will continue to drip water from projecting points for many hours after the rain has ceased. Solution of the cement paste by the rainwater percolating downward within the staves will eventually cause serious structural damage.

It is believed that staves having permeabilities of zero as determined by the method described in reference[2] should be safe against damage by percolation. It was shown in reference[1] that staves meeting the strength requirements of the proposed stave specification[2] also had zero permeabilities, and consequently the strength requirement is sufficient to prohibit the use of excessively porous staves.

Acid action, though not the major factor, has an important influence on silo durability as will be shown later. For the moment, considering only the interaction between the silo joices and the wall materials with no frost action involved, unless it is assumed that movement of liquids occurs over long periods at the face of the walls, the quantity of acid coming in contact with the wall is not great enough to cause extensive damage. Even when acid is available in large quantities as in laboratory tests or when silage joices run down the exteriors, the damage is to the

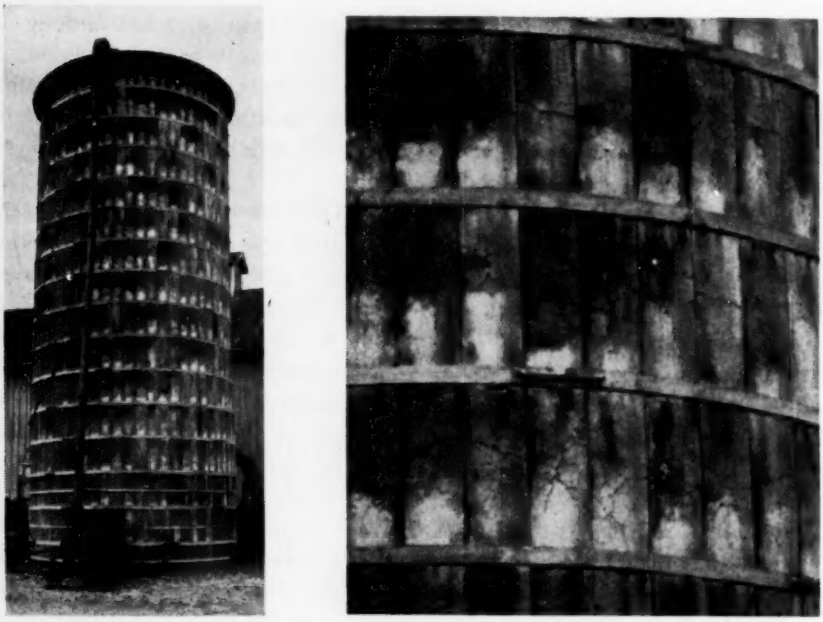

Fig. 6 (a)—Dry-tamped stave silo showing marked disintegration due to frost action. (b) Detail showing typical frost disintegration of dry-tamped staves.

surface only, the body of the stave remaining hard and strong[9]. Thus the structural damage of acid is slight even under the most favorable conditions for its action. The senior author has seen only one silo badly damaged by acid in spite of earnest efforts to find such silos. After thorough examination of this case, it was his opinion that the silo could be put in service again for many years by the replacement of a few staves, proper adjustment of hoops, and treatment of the interior to close the joints and produce a smooth surface. The damage in the first case was largely due to neglect of proper maintenance.

Accepting the conclusions that (1) acid action is a surface action (2) frost action is a surface action when the interior surface is more resistant to water movement than the body of the stave and (3) both (1) and (2) are unavoidable but do not result in serious structural damage if the silo is given adequate upkeep, it follows that short lived silos can be avoided by eliminating those materials and methods of construction which lead to frost disintegration in the body of the stave.

Fig. 5, 6, and 7 show silos which have suffered disintegration due to frost action and also serve to illustrate some of the points previously discussed. The photographs and quotations are from a silo survey made by Dalton C. Miller[9].

Fig. 7—Interior surface showing frost scaling

The silo of Fig. 5 was of monolithic construction and had been in service 28 years when the photograph was taken in 1937. It will be seen that the disintegration is most obvious on the roof, the chute, and the upper part of the walls. The aggregate used in its construction was reported to have a large clay content because of improper stripping of the gravel pit. It is evident from the photograph that steel forms were used. Because ventilation of the barn was through the chute, both the temperature and humidity within the chute and over the silage would be higher than that of the outside air and hence vapour pressure differentials would exist. Due to the uncontrolled and large clay content high capillarity and low strength concrete might be expected. Spading against the steel forms resulted in a dense outer surface thus forming a moisture barrier on the cold side of the stave. The combination of the above factors will cause moisture gain in the body of the concrete and saturation just behind the outer skin. The result of the ensuing frost action is shown in the photograph. It is obvious that acid action had little to do with this failure, particularly in view of the statement made in the inspection report that "the inside is said to be in good condition".

The silo of Fig. 6 was built from dry-tamped staves and had been in service for 15 years when the photograph was taken in 1936. Close examination of Fig. 6a and the enlarged section of 6b will show that the staves are badly checked and cracked. The staining is believed to be due largely to calcium salts from cement dissolved and deposited by rain action. The horizontal cracks between the hoops seen in Fig. 6a are caused by flexural stresses in the stave set up by rod tension created by circumferential expansion in the staves due to frost action. In the

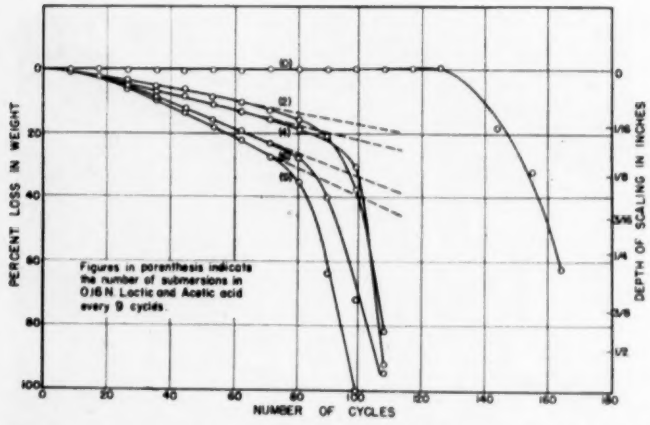

Fig. 8—Effect of various acid exposures on the resistance to freezing and thawing

inspection report it is stated that the "silo will probably fail from physical failure of the staves before acid disintegration would cause failure".

Fig. 7 shows the interior of a wet cast stave silo which had been in service 7 or 8 years when the photograph was taken. Examination of the photograph will show that typical frost scaling is occurring on the interior face at the bottom of the silo. The inspection report states "some staves have lost ½ in. of surface inside". This photograph was included to show that frost action does occur in the areas at the bottom which is reputedly subjected to the most severe acid action. The exterior of this silo also shows extensive frost action.

Though the above reasoning and evidence have convinced the authors that frost, and not acid, is the chief factor affecting the life of a silo, properly built and kept in adequate repair, the data of Fig. 8 show that acid action must be included in an acceptance test for durability. The specimens used to obtain the data of Fig. 8 were 2-in. cubes made from a plastic mortar having the following characteristics: mix by dry weight 1:3.18, water-cement ratio 5.25 gal. per sack, 5 minute absorption 2.4 per cent, 24 hour absorption 6.6 per cent, compressive strength after 7 days moist curing and 7 days air curing, 7920 psi. The aggregate was high in chert. The cubes were subjected to the 4-cycle varied by acid exposure of 0, 2, 4, 6, and 9 submersions of one-half cycle each per each 9 cycles. The plotted data are each averages for three specimens.

Referring to Fig. 8, the per cent's loss in weight represented by the straight line portion of the graphs for the specimens treated with acid are due to acid action and increase with increase in exposure to acid. Frost disintegration is manifested by the curved portions of the graphs. It will be noted at once that the frost resistances of the specimens sub-

jected to acid are markedly less than the resistance of the specimens subjected to frost action alone. It will be noted further that the points of tangency of the straight lines and the curves are all at about the same number of cycles regardless of the degree of acid exposure. Hence, it can be inferred that even slight acid exposure will cause a noteworthy decrease in frost resistance. Consequently any test for stave durability must include exposure to acid. It is also evident from Fig. 8 that if the acid exposure is severe and the end point is taken at a low per cent loss in weight, the test will evaluate the acid resistance but give no data on the effect of the major factor, frost action.

The authors are of the opinion that a durability cycle similar to the P-cycle but including one immersion in 0.16*N* lactic and acetic acid solution for each nine cycles (once per week) with an end point of 50 per cent loss in weight holds the greatest promise for a satisfactory durability acceptance test.

Such a cycle referred to for convenience as cycle F has been in use in this laboratory for some time, but no data on dry-tamped staves are yet available which will permit a fair estimate of the number of cycles required to reach the end point. The average per cent loss for 16 dry-tamped specimens at 112 F-cycles is five per cent, whereas the average number of 4-cycles at 30 per cent loss in weight for companion specimens is 79. Obviously, the number of F-cycles will be considerably in excess of the number of 4-cycles.

Examining cycle F by the criteria set up for an acceptance test, it is believed to approach requirement (1) more closely than any test cycle previously tried in this laboratory. It satisfied requirements (2) as to range and distribution of data (data of the 4-cycle was used for this conclusion) and requirement (4) as to simplicity of procedure and availability of apparatus. From data on similar cycles, it is believed that cycle F will satisfy requirement (5). However, it does not satisfy requirement (2) as to the time required for the test. Roughly estimating the average number of cycles required for a 50 per cent loss in weight and point as 200, the test would require 5 or 6 months. The cost of running individual specimens obviously would be prohibitive. However, if say twenty or more manufacturers submitted samples to one laboratory at the same time the cost per manufacturer would be reduced to reasonable proportions. The test need not be applied to the product of a given plant oftener than say, every second year provided that no radical changes in materials or methods of manufacture are made.

Accumulation of data from such tests would, in time, permit reasonably accurate estimates as to the effect of proposed changes in materials or methods of manufacture.

It may seem at first reading unreasonable to suggest that such a lengthy test be required by specification. However, if it is remembered that an attempt is being made to estimate the behaviour of a structure after 30 years or more of service, 5 or 6 months of testing may appear too short rather than too long. The authors are rather strongly convinced that acceleration of durability tests can be overdone. Those manufacturers who have been unfortunate enough to market non-durable staves or the equally unfortunate purchasers will recognize that if the test functions it would be a real money saver. The cost of one or two silo replacements will pay for a considerable volume of routine testing.

ACKNOWLEDGMENTS

The authors wish to acknowledge assistance from the following sources: D. G. Miller for supplying the specimens, the short time data thereon, and the photographs of Fig. 5, 6, and 7; the Graduate School and the Engineering Experiment Station of the University of Minnesota for the funds and facilities used in making the tests and the National Youth Administration in supplying student help.

REFERENCES

1. "Proposed Specifications and Methods of Testing for Concrete Staves to be used in Farm Silo Construction," ACI JOURNAL, Feb. 1940; *Proceedings* V. 36, p. 401.
2. "Permeability, Acid, and Absorption Tests of Mortars used in Dry-Tamped Silo Staves." C. A. Hughes, ACI JOURNAL, June, 1940; *Proceedings* V. 36, p. 553. Also Technical Paper No. 19, Engineering Experiment Station, University of Minnesota.
3. "A Flow Method for the Determination of the Effects of Soluble Chemicals on Concrete." Dalton G. Miller, Charles F. Rogers, and Philip W. Manson, Proc. A.S.T.M. V. 39, 1939.
4. "Standard Specifications for Drain Tile." A.S.T.M. Standards, 1940, pt. II.

5a. "Pavement Scaling Successfully Checked." O. L. Moore, *Engineering News-Record*, Oct. 10, 1940.

5b. "More Durable Concrete with Treated Cements," M. A. Swayze, *Engineering News-Record*, June 18, 1941.

6. "The Effect of Fine Aggregates on the Durability of Mortars," C. A. Hughes and Kenneth A. Anderson, Proc. A.S.T.M. V. 41, 1941.
7. "Tests on Concrete Masonry Units using Tamping and Vibration Molding Methods," Kurt F. Wendt and Paul M. Woodworth, ACI JOURNAL, Nov. 1939; *Proceedings* V. 36, p. 121.
8. "Thermal Conductivity and Surface Teatment of Silo Walls," Henry Giese, Bul. No. 303, Ag. Experiment Station, Iowa State College of Ag. & Mach. Arts.
9. An unpublished report to the Portland Cement Assoc. by W. G. Kaiser and others covering a survey of 125 silos and an extensive laboratory investigation of acid and frost action on mortars and the efficiency of surface treatments.
10. An unpublished report on silo inspection made over several years by Dalton G. Miller. Its excellent photographs and service records of over 100 silos make it the most important contribution to silo durability within the authors' knowledge.

Discussion of this paper should reach the ACI Secretary in triplicate by Apr. 1, 1942, for publication in the JOURNAL for June, 1942

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AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN (Supplement) November, 1942

Discussion of a paper by Hughes and Anderson:

Observations on the Durability of Dry Tamped Silo Staves*

DISCUSSION by R. E. COPELAND, DALTON G. MILLER and AUTHORS

ERRORS IN ORIGINAL PAPER

p. 239, Last two sentences of first complete paragraph should read: "In another investigation no correlation was found between the H-cycle and natural weathering nor was any relation found in this investigation to establish confidence in the usefulness of data from the H-cycle. Consequently no data for either the A-cycle or the H-cycle are shown." (to clear up erroneous reference to "P-cycle").

p. 248, first sentence should read in part ". . . the body of the stave remaining hard and strong [2]"—this to correct the reference.

p. 248, last sentence should read: "The photographs and quotations are from a silo survey made by Dalton G. Miller[10] (to correct Mr. Miller's middle initial and the reference symbol).

BY R. E. COPELAND†

The authors of this paper have done well to focus attention on the possible importance of frost action in the disintegration of concrete silos.

However, it may be questioned whether the data presented are sufficiently extensive to warrant the conclusion that acid attack is a secondary factor compared to frost action.

There is no question that silage juice attacks concrete rather rapidly when it is present in such amounts as to flow down the silo walls and through the joints. On the other hand, when the mositure content of the silage is properly controlled, experience indicates that there is very little trouble either with corrosion or joint leakage. The writer's observations lead him to believe that the problem is definitely related to the amount and mobility of the silage juice and that much of the disintegration, whether it be due to frost action or acid attack, can be eliminated by adopting ensiling methods which avoid an excessively wet

*ACI JOURNAL Jan. 1942; *Proceedings* V. 38, p. 237.
†Portland Cement Assn, Chicago.

silage. Not only is the excess moisture injurious to the structure, but unless drained away by some means, impairs the quality of the silage.

The theory presented in the paper concerning the moisture and exposure conditions in silos and their probable influence in promoting frost action seems to be logical but should be more definitely verified. It would seem that if the theory is correct it would apply generally and all of the thin wall type silos should be affected. This does not agree with the experience of several observers, that without an actual flow of juice the average stave silo is not affected to any considerable extent over a period of years.

The relative importance of different factors is obscured by the many inconsistencies which cannot be explained adequately for the lack of complete data. For example, in Fig. 5 of the paper, it is not clear just why certain areas of the silo are badly disintegrated while adjacent areas, which obviously were subjected to the same exposure, appear to be in perfect condition. It is doubtful if such isolated examples are very helpful unless such an inconsistency can be explained. Should it be ascribed to differences of exposure or of concrete quality?

In Fig. 6 of the paper is shown another example of what is definitely called frost disintegration. While frost action may have been a causative condition, the possibility cannot be ignored that the disintegration shown would have developed with natural weather exposure and is not related to conditions particularly identified with silos.

Referring to Fig. 7, it is not clear how this disintegration can be positively ascribed to frost action. Could it not be due to acid attack alone or to a combination of acid and frost action?

With reference to some of the test results, Fig. 2 for instance, it is suggested that a wide variation in individual values is to be expected. Trends and relations would be more definite if each plotted point represented the average of at least six cube specimens.

The test proposed by the authors should be useful to investigators and stave manufacturers but would appear to be impracticable as an acceptance test because of the time required.

The authors are to be commended for injecting some fresh thinking into the problem and it is hoped that they will find opportunity to develop more evidence on the subject. More extensive and thorough investigations of actual structures would help to clarify some questions, not the least important of which is the one concerning the real seriousness of the corrosion problem. The writer is inclined to the opinion that there has been a tendency in recent years to somewhat exaggerate the problem.

By DALTON G. MILLER*

This paper is based on careful and extensive laboratory studies and is an outstanding contribution to the subject considered. The authors are to be commended for focusing attention on the importance of resistance to weathering if concrete silos are to be truly durable. They are further to be commended for showing the inter-relationship of silage juices and frost as agencies of deterioration. However, after reading the paper, the general impression that remains with this writer is that the data presented emphasize that high quality dry tamped staves displayed great durability under the tests, and, considered from a practical standpoint, it is unfortunate that this was understressed by the authors when the fact is so clearly indicated.

Referring to Fig. 1 the authors state ". . . the absence of a relationship clearly indicates that the transverse strength is not an adequate criterion of resistance to freezing and thawing." In Fig. 1, it will be seen that there were a total of 53 staves tested. Of these 53 staves, 25 passed the specification strength limit indicated, and 28 did not meet the requirement. Of the 25 that met the strength requirement 20 (80 per cent) had not failed at 800 reversals while of the 28 staves that failed to meet the strength requirement, only 5 (18 per cent) passed 800 reversals.

The authors stated early in the paper that, "The investigation has been predicated on the belief that the essential requirements of an acceptance test are:

1. "The test is a true measure of quality under examination." They further state, "Because of the time element involved in durability tests and service records, it is doubtful if there ever will be general agreement that any acceptance test for durability satisfies the first requirement." Now bear in mind that the staves from which these cubes were cut were from a number of manufacturing plants, of aggregates from different sources, of different mixes as regards the quantity of cement, variously tamped and cured, and of different ages when tested. Regardless of all these variables the correlation between transverse strengths and durability, while short of the stated requirements of an ideal performance test, was of a high order and the test has one fundamental requirement of any completely satisfactory performance test in that it is easily applied. Much may be said in favor of any fairly reliable performance test that is simple enough to be readily applied, and as a consequence generally used, as against a more accurate test that is so tedious of application that it is rarely used.

*Senior Drainage Engineer, Soil Conservation Service, U. S. Department of Agriculture, University Farm, St. Paul, Minn.

Referring to Fig. 2, the authors state that "no relation between acid resistance and frost resistance was obtained for those specimens which had reached an end point". The statement may be true in a strict sense, but there is ample room for difference of opinion regarding the significance of the data as presented in Fig. 2. Results of tests of 52 staves are shown on the figure of which 23 had not reached the end point of 800 reversals of freezing and thawings. Of these 23 staves, 19 (80 per cent) required more than 75 days under the acid test to scale to a depth of $\frac{1}{8}$ in. Of the other 7 staves that required 75 days to scale to a depth of $\frac{1}{8}$ in., 4 went in excess of 550 reversals before failure and the 3 others were by no means the worst of the lot of all those that failed early in the frost test. Correlation between the two tests is admittedly not exact, but there is a definite general agreement.

Referring to results of the 5-minute absorption test as shown in the upper left corner and in the upper right corner of Fig. 3, the authors state, "No relations are exhibited by the data for either the five minute absorption or for the ratio of five minutes to 24 hours, though it is true the chance for high resistance to frost action is greater for specimens having low values of either these measures of absorption." Here again it is entirely logical to arrive at exactly the opposite conclusion. Inspection of the fifty 5-minute absorption points as plotted at the upper left of Fig. 3 shows that 20 staves had passed 800 reversals of freezing and thawing of which 16 (80 per cent) were below 3 per cent absorption at 5 minutes and 2 more very slightly above this limit. On the other hand, of the 30 staves that failed, only 6 (20 per cent) were below 3 per cent absorption. Considered alone the 5-minute absorption test of the staves shown would not be entirely satisfactory as a performance test, but certainly there is definite correlation of results of this test and the resistance of exposure to the P-cycle.

The authors state, immediately following Table 1, that "the surprisingly high resistance of the dry-tamped mortars to the P-cycle is attributed to their high air voids, etc.," and mention that "Durability studies on brick have shown that the effect of freezing and thawing is greatly reduced if the ratio of absorbed water to pore space is 0.8 or less." The writer questions that dry tamped silo staves of high strength and low permeability can fairly be said to have high air voids, as this is a characteristic of low strength staves of lean sand mixes of consistencies on the dry side. As regards the reference to the permissible saturation coefficient of 0.8 for brick, A.S.T.M. Specifications for Clay Building Brick (C62-39T) contains this clause under 2 (c), "If the average compressive strength is greater than 8000 psi. or the average water absorption is less than 8 per cent by weight after 5 hours submersion in boiling water, or less than 6.4 per cent in cold water, the

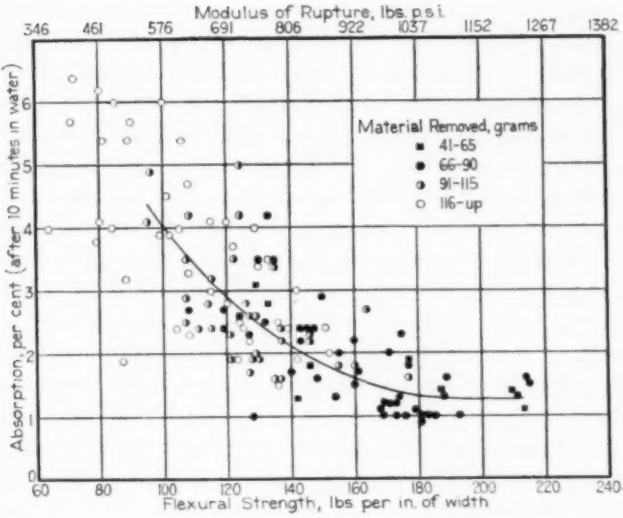
Modulus of Rupture, lbs. p.s.i.
346 461 576 691 806 922 1037 1152 1267 1382
Material Removed, grams
41-65
66-90
91-115
116-up
Absorption, per cent (after 10 minutes in water)
0 1 2 3 4 5 6
60 80 100 120 140 160 180 200 220 240
Flexural Strength, lbs. per in. of width

Fig. A

requirement for saturation coefficient (C/B) shall be waived." In other words, the high strength dense brick of low absorption are assumed to be weather resistant without further test, an assumption that exactly coincides with the opinion of the writer as regards durability of high strength, low absorption concrete silo staves. Both clay and concrete products of this type are weather resistant to a high degree.

At University Farm of the University of Minnesota tests have been made of many concrete silo staves during the past 15 years. Included in the tests was one of exposure to weak solutions of acetic and lactic acids.*

Results of tests of 126 silo staves from some dozen plants are shown on the accompanying graph (Fig. A) of which the ordinates are strength and 10-minute absorption. The staves have been separated into four groups as indicated based on loss of material during 20 days exposure.

Reference to this figure shows that those staves which best resisted the acid were in general those high in strength and low in 10-minute absorption. If a minimum limit were fixed at 140 lb. per in. of width for the transverse strength, and 2 per cent for the maximum absorption after 10 minutes in water, 38 staves would have met requirements 33 of which (87 per cent) were in the two groups highest in acid resistance, while only 2 staves (5 per cent) from the group lowest in acid resistance would have passed. If the strength requirement be kept at 140 lb. and the absorption limit raised to 3 per cent, then 50 staves would have

*Laboratory Tests of Concretes and Mortars Exposed to Weak Acids, Miller, Manson, and Rogers, Agricultural Engineering, Nov. 1939 (V. 20, No. 11).
A Flow Method for the Determination of the Effects of Soluble Chemicals on Concrete; Miller, Rogers, and Manson, ASTM Year Book, V. 39, 1939.

passed of which 40 (80 per cent) were in the two groups most resistant to acid and but 4 staves (8 per cent) from the group lowest in acid resistance. At the other extreme, of 29 staves with strengths under 120 lb. and absorptions in excess of 3 per cent 24 (86 per cent) were from the group lowest in acid resistance while the others all came from the group second lowest in resistance. Most certainly these results indicate correlation between strength and absorption and acid resistance. Admittedly the correlation is not exact, but the writer believes well within usable limits. A 10-minute, or other short time absorption test is closely reproducible regardless of the age of the specimen and is a fair indicator of permeability, provided the size and general shape of specimens are reasonably uniform as may be specified. The transverse strength of staves has been found somewhat erratic although generally not so to an extreme. It would be better laboratory practice to make three breaks of each stave and take the average as the true transverse strength. The routine would be to first break the stave by loading at mid point of a 24-in. span and then break each half by loading at mid point of a 12-in. span, reducing each short span result by one-half.

Now it is true that some silos built of staves of relatively low quality as indicated by strength and absorption tests have had good service records. On the other hand, the writer has never seen a silo of high strength and low absorption staves, made of durable aggregates, that gave any evidence of distress because of weathering. Exposure conditions of individual silos vary widely due to a number of factors among which is the moisture content of the material ordinarily ensiled. This varies greatly due to local practices. Corn grown on peat land in the northern latitudes is generally immature when ensiled making for wet silage while corn from high land is in many cases ensiled on the dry side, with respect to best keeping qualities of the silage, but decidedly favorable with respect to acid action on the silo. Some barns ventilate the moist barn air through the silo by way of the chute, a situation extremely severe for low quality concrete. Because of the wide difference of exposure conditions to which silos may be subjected it serves little useful purpose to cite low grade concrete silos that have given many years of good service as evidences of durability because the same structures conceivably would have given very unsatisfactory service under more severe exposure conditions. A permeable silo stave, even though it might last a long time in service, is not a satisfactory unit because it permits passage of discolored silage juices, and makes an unsightly structure of which there already are far too many.

It is stated in the paper, just before the discussion of Fig. 2, that "The authors know that very dense rich and strong mortars made with durable aggregates are highly resistant to freezing and thawing," and scattered

all through the literature on concrete are many papers dealing with the problem of weathering. The writer knows of no evidence that discredits well placed high strength concrete of durable aggregates. Because of the methods of manufacture in general use, it is not feasible to produce dry tamped silo staves of high strength and low permeability unless close observance be given the following details:

1. Well graded, durable aggregates. Stripped of all technicalities, this means well graded concrete sand plus about 20 per cent gravel passing a ⅜-in. screen, and retained on a No. 4, the size generally spoken of as "pea" or "buckshot," all materials to meet the ASTM specifications.

2. Consistency of mix as wet as will permit immediate stripping of the forms without undue "sagging" of the staves. The water cement ratio for commercial dry tamped staves, as made, can hardly exceed 4 gal. of water per bag of cement. Undue "sag" is about .030-in. out of line measured along either edge of the stave.

3. Rich mixes. High strength, low permeability dry tamped staves are rarely of a mix leaner than 1:4, about eight 65-lb. staves per sack of cement.

4. Adequate tamping and proper curing.

In conclusion, the only argument raised against making staves as rich as 8 per sack of cement is the cost. This is not a valid objection as can be shown. For instance, about 800 staves are required for a 14x40-ft. silo and made at the rate of 8 per sack, will require 100 sacks of cement. If made at the rate of 11 staves per sack, 73 sacks will be required. Assuming the cost of cement during normal times at 55 cents per sack, the saving of 27 sacks of cement will amount to $14.85. The complete cost of a 14x40-ft. concrete stave silo ordinarily runs between $600 and $800; therefore a difference in cost of $14.85 is less than 2½ per cent. If, as the writer believes, this slight additional cost increases the durability of the structure, the amount is of no consequence spread over an anticipated life of many years.

Dry tamped concrete silo staves made along these lines will have flexural strengths averaging upward from 130 lb. per in. of width and generally will have a 10-minute absorption under 2 per cent. Such staves will not wet completely through following storage in water for 24 hours. They will be durable against frost action, resist acid to a high degree, and remain at good appearance after many years of anything like normal service. Such staves are no better than are now made at many, probably most, of the plants in Minnesota and are truly a durable high grade concrete product.

AUTHORS' CLOSURE

Because the acceptance of frost action instead of acid action as the chief factor in silo disintegration requires radical revision of present

opinion on several features of silo construction, particularly on surface treatments, the authors had hoped for full discussion of the paper by those interested in the silo industry. Consequently, Mr. Copeland's contribution is very welcome.

In his second paragraph Mr. Copeland expresses doubt that the data of the paper justify the conclusion that frost action is more important than acid action in silo durability. In his last paragraph he suggests the desirability of "more extensive and thorough investigations of actual structures." As a matter of fact the paper presents no data leading directly to the above conclusion other than the photographs of Fig. 5, 6, and 7. It does present an explanation for the evidences of disintegration found in some silos in service of which those of the above figures were selected as typical. This explanation was reached after some years of extensive field and laboratory investigations and many hours of discussion with silo manufacturers and silo experts. No silo manufacturers of the authors' acquaintance will deny that at least 90 per cent of all concrete silos that have become total losses failed because of excessive spawling or cracking similar to that exhibited in Fig. 5, 6, or 7. Laboratory investigations leave little doubt that spawling and cracking are not due to acid corrosion, whereas they are generally accepted as manifestations of frost action.

Referring to Mr. Copeland's fourth paragraph, the authors do believe that their "theory" does apply to all thin wall type silos, and for this reason the photographs of Fig. 5, 6, and 7 were selected because they show frost disintegration for a monolithic silo and for silos built of dry-tamped and wet cast staves respectively. Comparison of the graphs of Fig. 8 of the paper for no acid exposure and for various degrees of acid exposure will show that acceptance of frost as the chief agent would "agree with the experience of several observers that without an actual flow of juice the average silo is not affected to any considerable extent over a period of years." All laboratory tests have indicated acid disintegration to vary directly with the time of exposure whereas a specimen exposed to freezing and thawing may show no evidence of distress for many cycles.

The inconsistencies referred to by Mr. Copeland in his discussion of Fig. 5 are not apparent to the authors. There can be no question, however, that the disintegration shown, particularly that in the chute, is not due to acid action. Frost action in chutes and roofs of monolithic silos is not as "isolated" as might be desired in this district.

In reference to Mr. Copeland's discussion of Fig. 6, the authors have always believed natural weathering to signify a disintegration caused by natural temperature variations in which frost action was the chief disintegrading agent. Consequently, their explanation of silo durability is in

perfect accord with the statement that "the possibility cannot be ignored that the disintegration shown would have developed with natural weather exposure and is not related to conditions particularly identified with silos." In other words, it was caused by frost action and not acid action.

With reference to Fig. 7, close examination will show scaling. While scaling is generally regarded as typical of frost action, it is incompatible with the acceptance of acid corrosion as a major factor in the disintegration.

So far as Mr. Copeland's discussion of Fig. 2 indicates the desirability of a greater number of data the authors agree with him. They do not believe, however, that as much information is shown by the use of averages as by the use of all data. The arithmetic average is generally regarded as a rather inefficient statistic.

The acceptance test proposed by the authors was intended for investigators and stave manufacturers and is obvisouly impractical as a test to be applied by a consumer to a given assignment of silo staves. If the investigators and manufacturers can be brought to agreement on an acceptance test for stave durability then, perhaps, some short time test adequate for the consumer's purpose can be devised.

[Note—About the time Mr. Copeland's discussion and the authors' closure were to have gone to press in the June 1942 JOURNAL according to schedule, Professor Hughes suggested postponing publication to admit discussion from Mr. Miller. Before Mr. Miller's discussion was received Professor Hughes had left the University of Minnesota to join the Canadian army and has not responded to the opportunity which ACI tried to give him to extend his original closure to include comment on Mr. Miller's contribution.—EDITOR.]

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Absorptive Form Lining*

By E. N. VIDAL† and R. F. BLANKS‡
Members American Concrete Institute

SYNOPSIS

Presents a summary of the development of the wallboard type of absorptive form lining as a practical means of eliminating defects and effecting other improvements in formed concrete surfaces. The paper describes briefly the laboratory investigations made, field tests to determine the practicability of the method, purchase specifications, and the experiences to date in using absorptive form lining in actual construction.

INTRODUCTION

For many years builders have been confronted with the problem of eliminating common defects such as air and water voids, sand streaks, watercourses, and unsightly joints on formed concrete surfaces. Attempts to develop a practical technique for improving formed surfaces, confined mainly to studies of mix proportions, consistency, compaction, form coatings, and special finishing procedures, have been only partially successful. The continually broadening use of concrete, not only as a structural material but also as a medium of modern architectural expression, created a growing demand for more pleasing and uniform appearance and for increased resistance to weathering and to high-velocity water flows in hydraulic structures. The Bureau of Reclamation began an intensive program of investigation in May 1938, which ultimately lead to the development of absorptive form lining well into the phase of practical use.

LABORATORY INVESTIGATIONS AND FIELD TESTS

The first studies[1] were made on concrete placed beneath an overhanging form, such as required for a sloping spillway surface. Observations and drill holes through pyralin windows in the forms conclusively showed that the surface pits were caused principally by entrapped air

(1)See Bibliography.
*Received by the Institute, Dec. 2, 1941.
†Engineer, Bureau of Reclamation, Denver, Colorado.
‡Senior Engineer, Bureau of Reclamation, Denver, Colorado.

and that movement of free water accounted for most other common surface defects. (These findings are in agreement with Earley,** whose pioneer work in developing architectural concrete to a high degree of refinement has been eminent for many years.)

The preliminary tests indicated the necessity for removing air and water accumulating at formed concrete surfaces and suggested the idea of an absorbent form lining as a possible practical solution. Burlap, muslin, fabric-covered screen and mesh, blotters, wood pulp, and other materials were tried with varying degrees of success. Best results in the laboratory tests were obtained with vacuum mats but the required procedure was not considered practical under complicated conditions.

The first real progress from a practical viewpoint was indicated by using ordinary insulating wallboards for form lining. Surface voids were materially reduced, other blemishes were diminished, and a general improvement in the quality of the surface concrete was noted. The major difficulty was occasioned by the board sticking to the concrete to a degree which required scraping or wire-brushing to remove embedded fibers. Tests with commercially available boards of various brands, types, and materials (wood, cane, and straw) demonstrated a common characteristic for bonding. Attempts to inhibit the sticking tendencies of wallboards were made by treating them with chemicals and oils. Linseed oil gave the most promising results but it was impracticable to apply the oil after the lining was installed in the forms, and treating the board beforehand or at the factory, created a dangerous fire hazard.

About this time, the use of fiber wallboard was given impetus by the experience gained during the construction of the Los Angeles Railway Terminal Building [2]. Standard fiber wallboard was used on that job for lining both exterior and interior wall forms. Uniform texture of the surface had been the original aim which, incidentally, was accomplished satisfactorily, but it was also observed afterward that the concrete surfaces were notably free of crazing and resisted disintegration from locomotive smoke better than ordinary formed concrete. Linseed oil was added on the job at least 48 hours before use, which effectively eliminated bonding to the concrete when the forms were removed within 5 days. (It is understood that a carload of material was lost by spontaneous combustion while stored after oiling.)

Laboratory efforts to develop fiber boards as a practical absorptive form-lining material were continued by applying muslin fabrics to the surface to prevent sticking. The laboratory results were very satisfactory as concrete free of surface defects was consistently obtained, with increased density, increased resistance to freezing and thawing and

**John J. Earley has presented many papers on this general subject, the titles of most being listed under his name in the 1941 ACI Directory.

abrasion, reduction of crazing tendencies, and higher flexural and compressive strengths as shown in Fig. 1 to 5, inclusive. It was obvious that the improvements in concrete qualities resulted from reduction in the water-cement ratio at and near the surface to provide a "case-hardened" effect[4] [5].

The problem of developing and fabricating a suitable material on a commercial basis was made possible through the splendid cooperation of several manufacturers of wallboard products. The use of fabric facing was soon abandoned as special chemical treatments to eliminate sticking and at the same time provide some control of the initial rate of absorption by the board proved more practical and economical.

Large-scale field tests were conducted in October 1939, on the downstream face of Grand Coulee Dam. Twenty-two different linings, furnished by four manufacturers, were tried under normal job conditions. Although the results were mediocre as compared to the results that are now being obtained, the field test at Grand Coulee Dam indicated that surfaces formed with wallboard lining were superior both in quality and appearance to those formed with wood or metal forms.[3] [6].

Friant Dam, where lining was first used on a practical basis, became a proving ground for many of the facts known about form lining today. Manufacturers were given an opportunity to demonstrate their products after potential effectiveness had been demonstrated in the laboratory. Practical handling and installation techniques have been developed and desirable performance characteristics established from the Friant Dam experiences[10].

Other construction agencies, notably the Tennessee Valley Authority [9], U. S. Corps of Engineers, and the Wyoming State Highway Department[8], became interested in absorptive form lining and conducted independent field and laboratory investigations or actually used form lining in construction. Johnson has presented an excellent report[9] on the investigations and experiences at Kentucky Dam. An important conclusion stated, in part, that: ". . . It is also highly probable that by increasing the water-cement ratio and reducing the cement content, the surfaces produced will still be considerably better than we are obtaining today with lower water ratios and higher cement contents . . . "

Laboratory studies are being continued for the purpose of establishing the fundamental, physical characteristics most desirable for a form-lining material, and the exact processes by which the indicated improvement in the quality of the surface concrete is effected. These studies have established the fact that absorptive form-lining materials now in use effects measurable decrease in water-cement ratio of the concrete to a depth of approximately one inch from the surface, depending on the

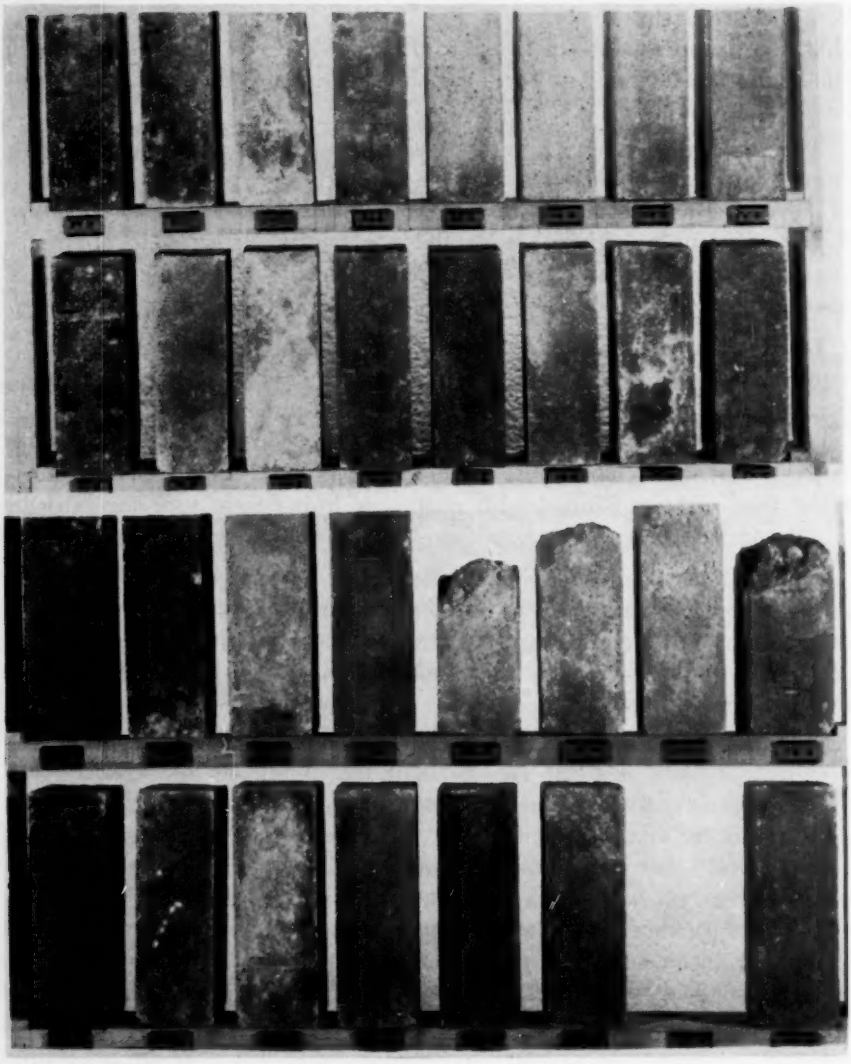


Fig. 1—Durability test on 3 x 3 x 9-in. concrete blocks formed by various types of lining as compared to wood-formed blocks (marked F W). The latter failed at approximately 110 cycles of freezing and thawing. Two top rows, 60 cycles; two lower rows, 100 cycles; two top rows, opposite page, 260 cycles.

The symbols designating specimens in the two top rows (after 60 cycles) are:
top row FF8, FC8, FH8, FN8, FW13, FW14, FW15, FW16
2nd row FF9, FC9, FH9, FN9, FF10, FC10, FH10, FN10
There is an identical sequence in the 3rd and 4th rows (100 cycles) and on the opposite page the 5th and 6th rows (260 cycles).

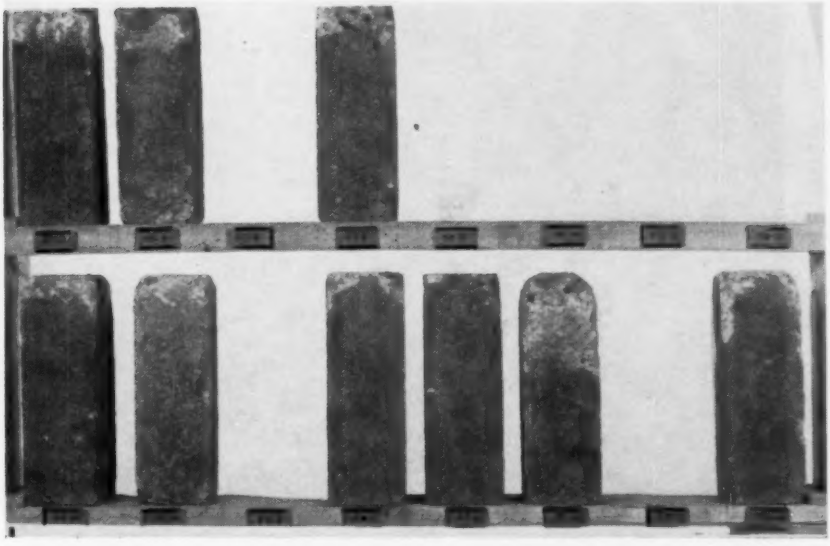


Fig. 1 Continued—See opposite page.

lining. The water-ratio decrease was the greatest at the surface, progressively reducing toward the interior as shown in Fig. 6.

This information was based on tests made by sampling sand mortars, two hours after placing, at varying depths from the linings and oven-drying each sample, to determine the water content. The reduced water contents near the surface are attributed to the absorptive action of the form lining.

SPECIFICATIONS

Specifications issued by the Bureau of Reclamation for the purchase of the wallboard type of form lining[7] were based principally on the physical properties (strength, absorption, adaptability for field use, etc.) of the board itself and a performance test in which a formed 3- by 5-ft. concrete slab was cast on a 0.7 to 1 slope. The physical properties were based on tests of boards which had functioned most satisfactorily in the previous tests. The purchase specifications also stipulated:

> The form lining shall be capable, through its absorptive capacity, of eliminating, to the extent specified, voids, pits, and other common defects from the surface of concrete placed against it. The form lining shall be highly absorptive to air and water and possess surface characteristics resulting in a satisfactory surface texture of the hardened concrete and a minimum amount of sticking upon removal of the forms. The lining shall be readily adaptable to cutting, fitting, and any other operation necessary in connection with its use. The lining itself, and any coating or treatment employed in its manufacture, shall be such as will not create discoloration or interfere with normal chemical reactions of the cement in the concrete.

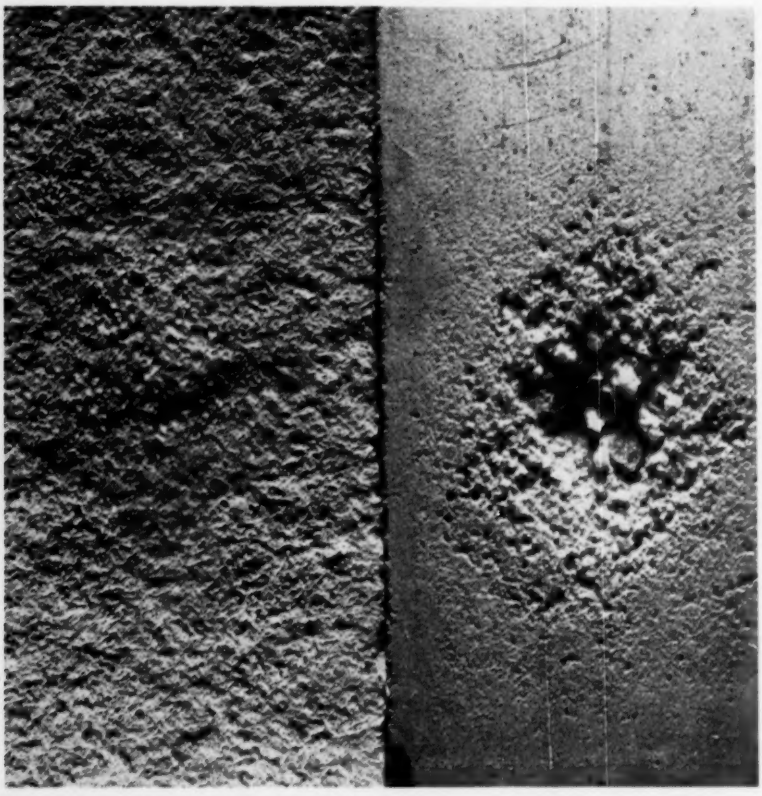


Fig. 2—One minute abrasion test. The surface of the wood formed concrete (right) was completely eroded in this test leaving nothing but the coarse aggregate to withstand the wear. The absorptive formed surface, (left), was only slightly affected by like treatment.

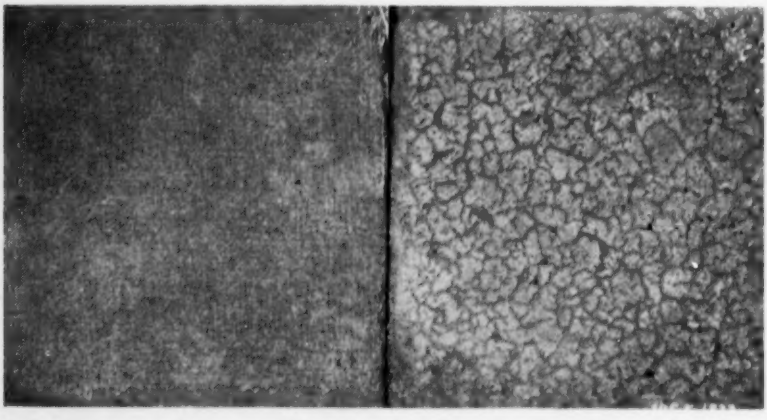


Fig. 3—The concrete surface produced against a wood form (right) crazed much more than the surface produced against an absorptive lining (left).

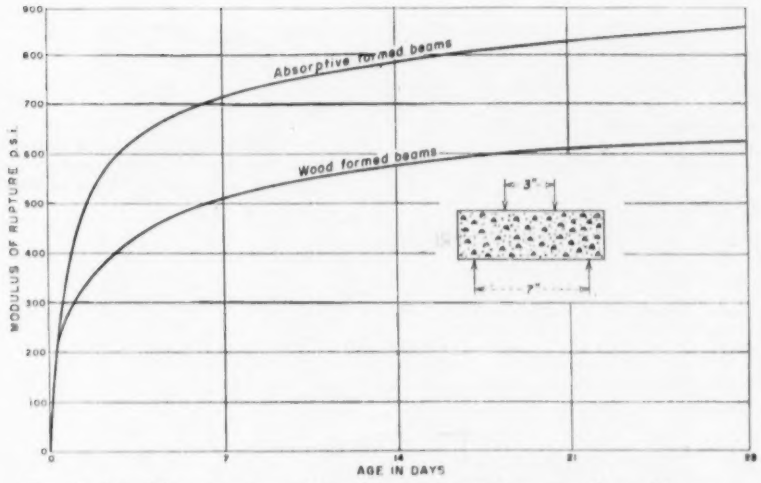


Fig. 4—Modulus of rupture of wood formed and absorptive formed beams.

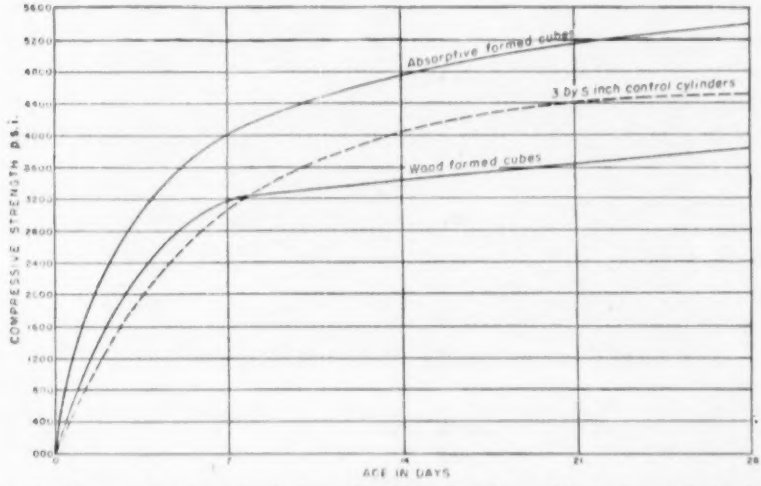


Fig. 5—Compressive strength of absorptive formed and wood formed modified cubes.

It was required that bid samples be furnished to determine conformance of the proposed material. Provisions were included to control the size of bundles in shipment to simplify handling operations.

After accidentally installing sheets backwards on some of the early work, the specifications were revised to provide for appropriate markings in every square foot of each sheet. The reverse side of form lining, in general, does not function in the manner desired.

Recently, attention and study have been directed to a different type of lining which consists of a thin absorptive paper-pulp sheet faced with

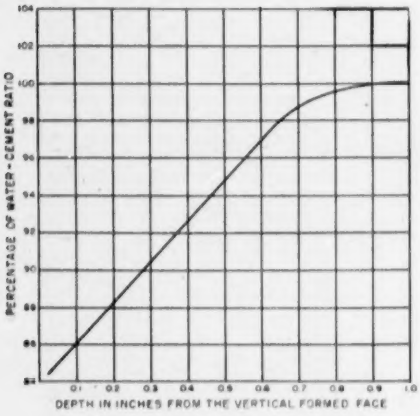


Fig. 6—Typical curve showing reduction in water - cement ratio of mortars formed against absorptive form lining.

muslin. The composite sheet is less than ⅛-in. thick and has been effective, both in the laboratory and in the field as a form-lining material.

FIELD EXPERIENCES IN THE USE OF ABSORPTIVE FORM LINING

During the last 18 months, several million square feet of the wallboard type of absorptive form lining have been used on a variety of Reclamation structures, including a large concrete dam, spillways and outlet works for earth dams, control houses, and power and pumping-plant buildings. The objectives have been to obtain: (1) Improvement in hydraulic characteristics and increased resistance of the concrete to cavitation resulting from high velocity water flow; (2) increase resistance to freezing and thawing and surface checking and crazing; (3) improvement in architectural appearance.

The results have been generally very satisfactory and the cost reasonable. The improvement in surface appearance has been striking, with practically all air voids, sand streaks, and other common defects eliminated (Fig. 7). Marked improvement in construction joints with the use of absorptive lining is also obtained, although when architectural considerations are of primary importance, special joint treatment, as discussed later, should be employed. While laboratory tests conclusively demonstrated the superiority in weathering resistance of absorptive formed over ordinary formed concrete, service histories are too short as yet to indicate the corresponding behavior under field conditions.

The delivered cost of the wallboard type of lining has varied from 3½ to 5 cents per square foot. The gross handling and installation costs vary with the particular job conditions involved, from 8 to 15 cents per square foot. In some cases a large part, if not all, of this over-all cost is offset by direct construction savings in the initial cost of forms, form cleaning, oiling and maintenance, curing and protection, and special surface finishing.

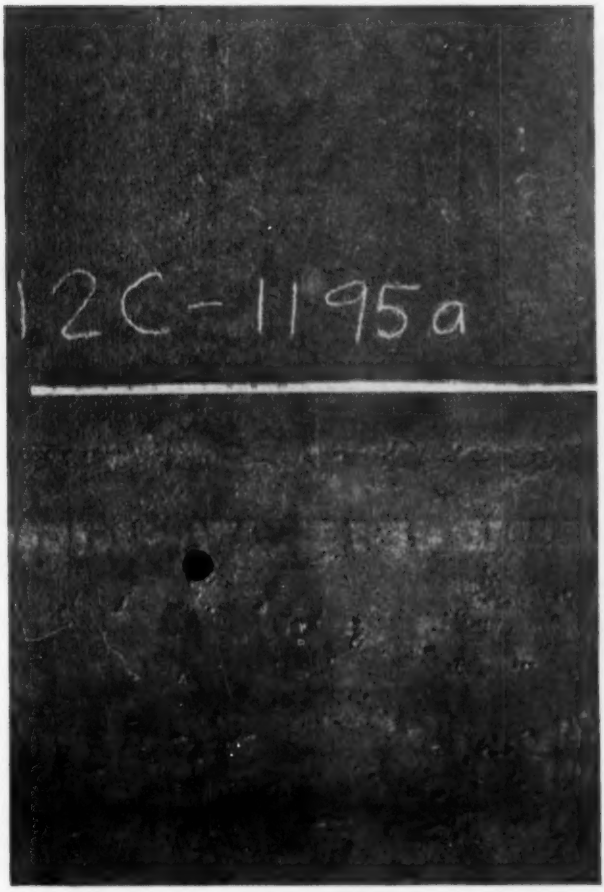
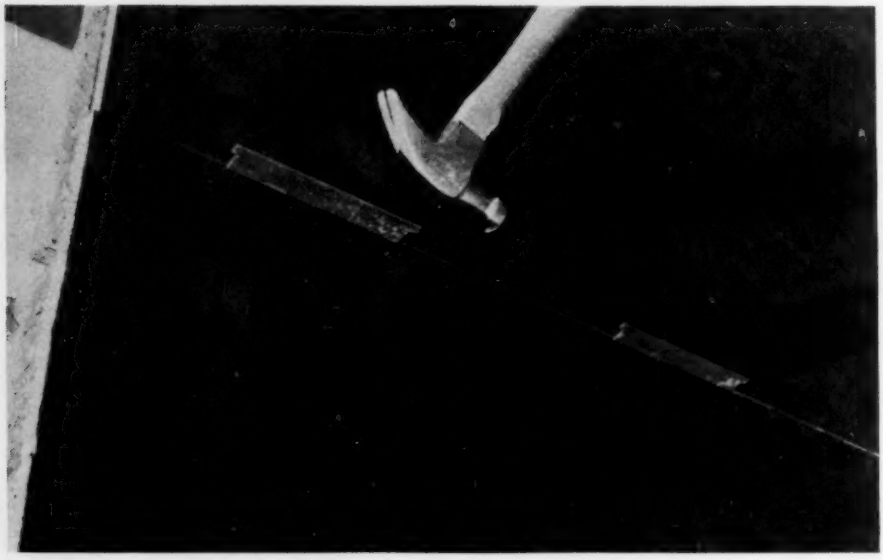


Fig. 7—Downstream face of a dam comparing surface formed by absorptive lining above with that produced below against ordinary wood lagging.

HANDLING WALLBOARD FORM LINING

Wallboard lining is normally supplied in sheets ½ in. thick by 4 ft. wide and 8 ft. long. Special sizes cut to predetermined lengths can be obtained readily but widths other than even factors of 12 feet usually require special manufacturing provisions or waste. A 4- by 8-ft. sheet has sufficient structural strength to permit ready handling by one man. Longer lengths can be obtained but they are somewhat awkward to handle and are more susceptible to damage during installation. The structural strength of lining boards now available limits the length which can be handled to about 12 ft. The boards should be obtained in bundles of six, wrapped in strong waterproofed paper for protection in handling



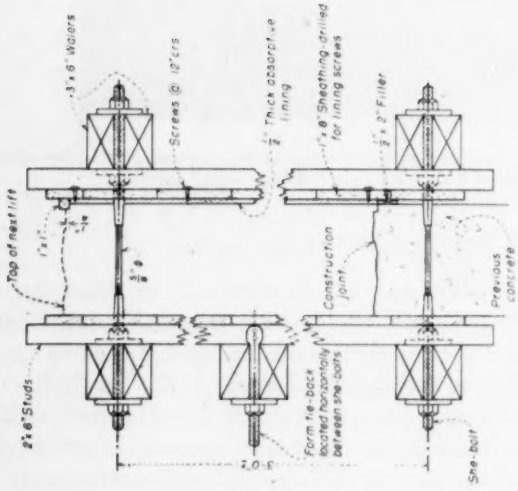

3" x 6" Waters
Screws @ 12" c/s
1" Thick absorptive lining
1" x 8" Sheathing drilled for lining screws
2" x 2" Filler
Top of next lift
Construction joint
Previous concrete
2" x 6" Studs
Form tie-back located horizontally between she coils
She-bolt

Fig. 8 (above)—Example of form construction proposed and used for thin wall structure. If desired nails from the lining could have been used in place of the screws.

Fig. 9 (right)—Metal shims for proper spacing of form lining sheets.

and from the elements until they are used. "Dog houses" or special crates should provide storage on the job for working supplies of boards.

INSTALLATION

On complicated work, layout diagrams showing the location and identification number of each board will be found advantageous. The sheets can then be cut to dimension in the carpenter shop or yard and quickly installed in the forms without confusion or delay after transportation to the work in the special crates or skips. Cutting can be satisfactorily accomplished with ordinary power saws. Holes can be drilled with common wood bits, or a cutter similar to a cork borer driven by a power hand drill has been found useful for drilling tie-rod holes. Other special cutting tools are available or may be devised and made where expedient.

Forms should be constructed in the usual manner with proper allowance in dimensioning for the liner sheets. Lagging need not be as smooth as for forms which are not to be lined. The lining should not be counted on to supply any structural strength or handling stability. Backing for the lining (lagging) should be of solid wood and reasonably free of irregularities. Fastening the lining to the form panels should be the last operation before final alinement and clamping the forms in place. Even then, care must be exercised to prevent damage to the lining in the final stages of preparation for actual concrete placing.

Fourpenny casing nails, driven just flush with the board surface, have been found adequate for fastening the lining to the forms. The nails should be spaced on 3- to 6-inch centers around the edges and on 12- to 18-inch centers over the balance of the lining surface. Nails with larger heads will leave imprints in the concrete. On one job involving thin walls, the lining was fastened with wood screws from the back side of the forms (Fig. 8). Form spreaders should be used sparingly and where used should be provided with large bearing plates to prevent compressing the lining. Absorptive lining expands considerably on absorbing moisture and the sheets should be spaced about $\frac{1}{16}$ in. apart to prevent buckling. Metal shims (Fig. 9) of proper thickness have been used effectively for spacer gages. With proper attention to the spacing detail, joints between lining sheets are hard to detect on the hardened concrete.

It is impractical to place absorptive lining during heavy rains but during light rains or after installation, protection with canvas has proved practical. Slightly damp lining can be dried after it has been installed by portable warm-air blowers which are available on the market.

The absorptive lining should overlap the old concrete about two inches at a plain construction joint. A filler strip of hardwood should be tacked to the form lagging just below the lining to provide bearing against the

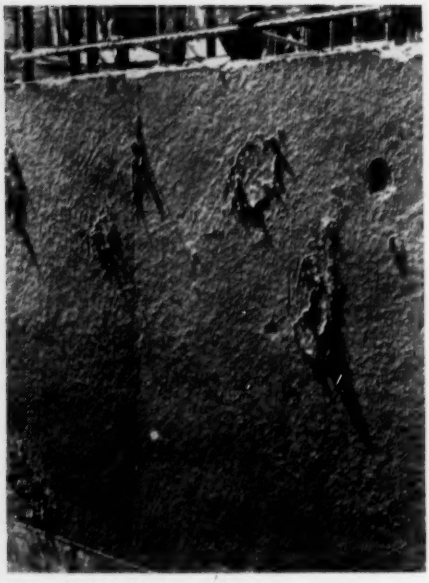


Fig. 10 (above)—Successful use of 1 - inch deep architectural grooves. A horizontal construction joint is located in the lower groove. Note the excellent surface texture of the concrete wall.

Fig. 11 (left)—Probably an extreme example of the resulting concrete surface when the fiber lining was damaged by contact with carelessly handled vibrator.

old concrete when the forms are tightened in place. The thickness of the filler strip will vary somewhat with the job conditions, rate of placing, and consistency of the concrete. The thickness should be such that the pressure of the fresh concrete will just balance the swelling tendency of the lining board as it absorbs moisture, and thus prevent bulging or an underlip at the construction joint. For dry concrete in massive sections where an appreciable interval elapses between placing successive layers, the filler strip should be the same thickness as the lining. For thin walls involving more rapid rate of vertical progress in placing the

wetter concrete, the filler should be somewhat less than the lining thickness, such that the lining board will be compressed about 1/16-in. when the forms are tightened in place.

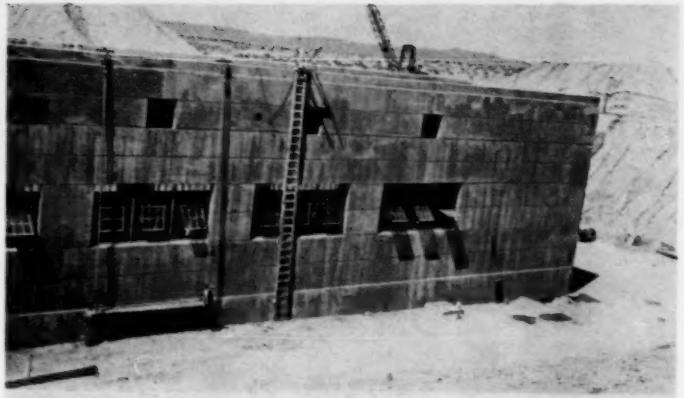
Although marked improvement in appearance at construction joints is obtained by the use of absorptive form lining, the joints will still be conspicuous on the hardened concrete surface. Therefore, where architectural effects are desired, grooves should be formed in the concrete at construction joints (Fig. 10). With a little thought and planning during the designing stage, architectural grooves can be used very effectively to obtain pleasing appearance.

PLACING CONCRETE IN ABSORPTIVE LINED FORMS

The final preparations for placing concrete after absorptive lining is in place, such as construction-joint clean-up and washing, installation of embedded parts, and placing of reinforcing steel, should be held to a practical minimum in order to avoid damage to the lining. No experience has been had in compacting concrete in absorptive lined forms by hand methods. The practicability of hand methods is questionable because field tests have shown that even specially designed tamping and spading tools will damage the lining wherever contacted and leave blemishes on the concrete surfaces. Internal vibrators have been used for compacting all concrete placed in lined forms on Reclamation projects. The vibrators must not be allowed to contact the lining (Fig. 11), or to come sufficiently close to kick the largest pieces of aggregate against the lining or the concrete surfaces will be blemished. When the heavy-duty vibrators are being used, as with mass concrete, a small, easily handled unit should be provided for compaction near the forms (Fig. 12). Concrete placing should be maintained at as uniform a rate as practical in layers not exceeding 12 in. The interval between layers should be less than 30 minutes if possible.

Theoretically the more moisture that the lining board will absorb from the concrete before it attains initial set, the more beneficial will be the effect. However, certain practical considerations have a direct bearing on rate of absorption by the lining material. When very dry concrete mixes are used or in a very hot arid climate, a high initial rate of absorption will tend to cause a grab action and a thin layer of concrete adjacent to the lining will "freeze" to the surface. This prevents proper consolidation, tends to trap air bubbles just below the prematurely stiffened surface layer, and cause a skin peeling when forms are stripped, if an appreciable interval elapses between placing successive layers of concrete. Because of these possible conditions, concrete slumps less than 1½ to 2 in. are not recommended with absorptive form lining. It has been found practical to alleviate the grab tendency of the board, particularly when

Fig. 12 (above)—Compacting mass concrete in a dam. Note the use of a light vibrator near the form.



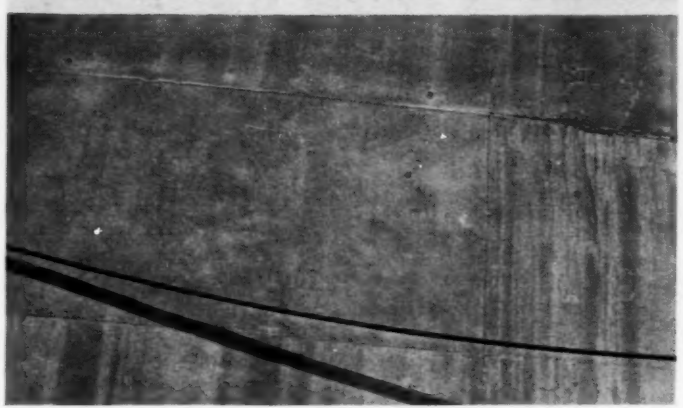


Fig. 13 (top right)—Curing water stains on partially finished structure. No final cleaning had as yet been attempted.

Fig. 14 (bottom right)—Elimination of curing water stains and other construction blemishes on a trial panel by sand-blasting.

caused by high temperatures, by lightly priming the lining just ahead of placing operations with a light fog water spray.

Vibration of the fresh concrete should be thorough and systematically accomplished. Mortar splash against the lining should be held to a minimum. If grooves are not used at construction joints, grade strips should be tacked to the forms in finishing out a lift to obtain a straight-line joint mark on the concrete surface.

STRIPPING FORMS

Forms should be stripped within 36 hours to avoid sticking of lining to the concrete. When forms are left in place for longer periods, fibers will adhere to the concrete surfaces. Wire brushing or light sandblasting is then required to remove the adherent fibers. The lining boards have practically no salvage value after one use as they have very little structural strength when saturated and cannot be removed from the job without being broken. Care in removing forms is required or the lining will be badly broken up, thereby introducing disposal complications.

TREATMENT OF CONCRETE SURFACES FORMED AGAINST ABSORPTIVE LINING

Concrete surfaces formed against absorptive lining present uniform appearance and an even texture. Any discontinuity of the normal appearance and texture is contrastingly magnified to a degree not encountered on surfaces formed with ordinary methods. Curing water stains (Fig. 13), patching, and construction marks of all kinds should be held to the practical minimum. If the utmost in architectural appearance is desired, the surfaces may be wire-brushed soon after forms are removed or very lightly sandblasted after completion of the curing period. In the latter method, a fine sand and low pressure should be used and the sandblast nozzle held at a flat angle. Recent observations have indicated that the sandblasting will be most effective if performed as soon as possible after curing has ceased. If this procedure is followed, the characteristic texture will not be affected (Fig. 14).

Patches may be made to blend unobtrusively with adjacent surface by firmly pressing or hammering a piece of the lining material against the freshly placed patching mortar. Matching color in the patching mortar can be obtained by blending white cement with the regular job cement in proportions as determined by trial.

CONCLUSIONS

1. Absorptive lining for formed concrete is a practical, proven method for eliminating voids and other common imperfections on formed concrete surfaces.

2. A uniform, pleasing surface texture may also be secured thereby. However, ordinary construction discolorations are abnormally conspicuous and should be removed if present on architectural concrete.

3. Absorptive-formed surfaces are superior in durability and resistance as compared to concrete formed with the conventional wood or metal forms.

4. It follows that concrete which will be placed against absorptive forms can be proportioned with a higher water-cement ratio and consequent decreased cement content for a given consistency and degree of durability.

5. The surface improvement which is gained by the use of absorptive lining is well worth the increased cost which can be offset to a considerable extent by cheaper form lagging, elimination of form cleaning or oiling, lower cement content, and elimination of special finishes.

6. Recent laboratory developments indicate that the improvement due to form lining is not limited to a skin effect but penetrates into the mass, being progressively less with depth.

7. Form-lining materials now available on the market would be materially improved if they were less susceptible to damage during normal construction operations and less affected by climatic changes.

8. More needs to be learned about the fundamental behavior and the basic physics of absorptive form lining and its action when used against concrete.

BIBLIOGRAPHY

1. C. O. Crane, "The Elimination of Voids on Formed Concrete Surfaces," Proceedings of the Highway Conference held at the University of Colorado at Boulder, p. 34, January 1940, republished in *Concrete*, p. 5, March 1940.

2. C. W. Pierce, "Architectural Concrete Surface for Los Angeles Terminal," *Engineering News-Record*, July 6, 1939.

3. "Absorptive Form Lining," *Western Construction News*, p. 41, February 1940.

4. William B. Cheek, "Case Hardening Concrete with Absorptive Form Lining," *The Engineers Bulletin*, Colorado Society of Engineers, p. 4, April 1940.

5. William B. Cheek, "Absorptive Form Lining," *The Engineers Bulletin*, Colorado Society of Engineers, p. 4, May 1940.

6. "Concrete Surfaces Improved by Absorptive Form Lining," *Engineering News-Record*, January 4, 1940.

7. E. N. Vidal and C. O. Crane, "Progress in the Use of Absorptive Lining for Formed Concrete," Proceedings of the Highway Engineers Conference at the University of Colorado in Boulder, p. 34, January 1941, republished in *Concrete*, p. 2, March 1941.

8. R. C. Pike, "Wyoming Uses Absorptive Form Lining" *Western Construction News*, p. 73, March 1941.

9. W. R. Johnson, "The Use of Absorptive Wall Board for Concrete Forms" ACI Journal, June 1941; *Proceedings*, V. 37, p. 621.

10. D. S. Walter, "Absorptive Lining for Forming Mass Concrete at Friant Dam," *Western Construction News*, p. 298, October 1941.

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Discussion of a paper by Vidal and Blanks:

Absorptive Form Lining*

CONVENTION DISCUSSION: MR. BLANKS RESPONDS TO QUESTIONS

Q.—Under existing priorities is this form lining material obtainable?

A.—We have had a little difficulty recently, although we have been able to obtain all that we have needed. It is not predictable how long this will continue.

Q.—How many uses can we get out of the material?

A.—It can be used once only, and has little, if any, salvage value.

Q.—What is the approximate cost per square foot?

A.—For the lining material only I think we now pay four cents a square foot. The installation cost will vary, depending on job conditions, anywhere from ten to fifteen cents per square foot for massive concrete work. For thin, heavily reinforced sections and where the ultimate in architectural appearance is desired, the cost may run as high as 35¢ per square foot.

Q.—What is your experience with concrete as cast face down—on top of the absorptive form lining as for a ceiling?

A.—We never made such use of the material but I see no reason why it would not function all right if sufficient care were exercised to avoid damage to the lining during placing and compacting of the concrete.

Q.—What are the bad features of the material?

A.—They are principally due to handling difficulties and the added complications on the job. There are no bad features, we have found, as far as results are concerned, other than that the lining has to be handled carefully to avoid damage and it must not get very wet. If damp it can be dried out. Under some conditions, it will expand—the

*ACI JOURNAL, Jan. 1942; *Proceedings* V. 38 p. 253.

joints between adjacent sheets must be correctly spaced. When the rate of absorption is too fast, it will tend to "freeze"—rapidly stiffen the surface skin of concrete. Under some conditions this may result in a surface peeling when forms are removed. There are a number of other minor complications when it is used on the job.

Q.—Is there any method of controlling the rate of absorption?

A.—The whole development of the wallboard type absorptive form lining was made possible through the splendid cooperation of a number of wallboard manufacturers, and they have been very active and very cooperative in developing the needed characteristics. Just what those characteristics should be is not as yet, any too well established, but we are continuing to learn something about it all the time. The rate of absorption can be controlled by surface treatment during the manufacturing process.

Q.—Does the use of form lining affect the stiffening time? Must you wait a certain time to prevent adherence?

A.—Stripping time varies with the job conditions, of course, and also with the particular type of board. With most linings that we have used, it is necessary to strip forms within 36 hours or the sticking is bad, but that is not true of all types of lining.

Q.—What is the experience on curved surfaces?

A.—That, of course, introduces complications, but the board will bend very easily around a curve of reasonable radius.

Q.—Might there be salvage value if the lining were dried out for reuse?

A.—No; it is gone after it is used once. It might be reclaimed and used for an insulating medium, but we have not found it worth while.

Q.—Does sand blasting spoil the texture?

A.—No, it does not. (Mr. Blanks referred to and showed illustrations of the paper.) You can see that the texture has not been destroyed or altered to speak of. Of course sand blasting has to be carefully performed; it should not be severe—a light pressure, and the nozzle should be held at a rather flat angle.

Q.—How about vibrators close to the lining during the placing of the concrete?

A.—That has to be watched carefully; vibrators should not be used close to the lining or they will force the larger pieces of aggregate against the form and leave a scar, or if the vibrator itself touches the lining, it will also leave a blemish. Surfaces formed against absorptive lining are so uniform in appearance with the absence of all the usual pock and form marks, that any blemish or stain or irregularity in the surface

shows up much more than on the ordinary concrete surface. When the weather is hot and dry, we have had to sprinkle the board a little with a very light spray, to prime it, so to speak, before we put the concrete against it, otherwise we have that "freezing" skin effect.

Q.—Is there more than one brand of absorptive liner on the market?

A.—Yes. Celotex, and Firtex are the two we have used the most. A new type has been developed by the U. S. Rubber Co., a blotting paper faced with muslin. We have not used that on any of our work yet; it has been used on some other jobs, I understand, and we will use it on one that is now starting.

Q.—How did you take care of rustication or chamfer shown in one of the illustrations?

A.—With just a wood furring strip. These grooves make a convenient place to locate construction joints, where they may be hidden very effectively.

Q.—How does the use of absorptive form lining affect the water-cement ratio, compared with ply wood and steel forms? Would you use more water in the mix?

A.—Absorptive lining reduces the water-cement ratio at the surface. We would use the water-cement ratio specified for the job, which would be the same for lined forms or ordinary forms.

Q.—What is the effect of using very harsh mixtures?

A.—I think the harshness would have no particular effect other than making it necessary to be more careful with vibrators. It is not practical to use concrete of too low a slump—than about an inch and a half. If it is drier than that, the "grabbing" effect of the absorptive lining on the surface concrete is more pronounced.

Q.—In sugar cane fiber, sometimes there is a little sugar left; have you had any surface trouble from that at all?

A.—No, we have not had that trouble. Those things were all discussed with the manufacturers at the time the materials were being developed in the factories. They understood the effects of different materials on concrete and made a particular effort to keep out those chemicals that would have deleterious effect.

Q.—Have you ever used absorptive lining in form vibration?

A.—We have not; as a matter of fact we rarely use form vibration except for supplemental tamping. We have used the clamped-on vibrators some in precast pipe manufacture.

Q.—Is it your opinion that it could be used?

A.—Yes, I see no reason why it could not.

Q.—Would it be correct to say that this is still in the stage of development?

A.—That is right; much is yet to be learned and much improvement to be made, particularly in developing the proper qualities that will make the board more effective, easier to use on the job, and perhaps so that it can be re-used.

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Transit Mixers Used Effectively on the Contra Costa Canal*

BY O. G. BODEN†

SYNOPSIS

This, the second of three brief papers each describing some element of construction operations on the Contra Costa Canal, considers the effective use of transit mixers on a nine mile section of the concrete lining. Five 4.34-cu. yd. capacity mixers delivered concrete to a canal lining machine which placed, vibrated and formed it in position—described in the first of these contributions.

Transit mixers were used effectively on a nine-mile section of the Contra Costa Canal where the limited width of right-of-way permitted only a narrow construction and operating road. Traffic problems were largely eliminated as other equipment was able to pass the transit mixers while they were discharging, except in the narrow places, where one way traffic was maintained. The transit mixers were always able to drive in and discharge their loads and then continue along the construction road to the nearest access road, instead of backing in as was often necessary with batch trucks and a crawler type mixer.

Five mixers were used, each with a maximum rated capacity of 4.34 cu. yd. of concrete when mixing in transit. The mixers were mounted on twin rear axle trucks with dual type wheels, and the mixer drums were powered by independent gasoline engines. Each mixer was equipped with a 150-gal. water tank, a motor driven pump, a visible dial water meter with an automatic shut-off, and a revolution counter to indicate the revolutions of the mixer drum.

The drum of each mixer was slightly inclined on its axis, with the high end to the rear of the truck, where it was charged through a feed chute. Mixing action was caused by large spiral blades in the drum and the discharge was effected by opening the door at the rear and reversing the rotation of the drum. The concrete could be observed through the discharge door to gage its consistency while mixing. A

*Received by the Institute, Jan. 29, 1941; see also "Machine Methods for Trimming Subgrade and Placing Concrete Canal Lining." ACI JOURNAL, Nov. 1941; *Proceedings* V. 38, P. 177.

†Construction Engineer, Bureau of Reclamation.

Fig. 1 and 2—Concrete delivered by transit mixers for lining Contra Costa canal.

number of mortar analysis tests were made on the job to determine the mixing efficiency of these transit mixers, using canal lining concrete with ¾-in. maximum aggregate and having 1½-in. average slump. The results of these tests indicated that the maximum variation was well within the allowable 10 per cent limit and that thorough mixing was obtained when simultaneous batching was used.

Cement, sand and gravel were batched at a central batching plant and wer charged simultaneously into the transit mixer. Mixing water, which was charged prior to, during, and following the mixer charging operations, was pumped from the tank through the meter which was set to shut off automatically at somewhat less than the desired amount. The mixer drum revolved during the charging operations, and mixing continued for a predetermined number of revolutions, several fewer

than the total required, while enroute to the point of placement. On arrival at destination, the consistency of the concrete was observed and more water was added as necessary to produce the required slump, provided that the total amount of water used did not exceed 0.60 W/C by weight, and mixing was continued as necessary to thoroughly incorporate the added water.

To obtain proper control of aggregates, frequent tests for surface moisture and grading were made at the batching plant and the batch weights corrected accordingly. Notations showing the maximum water which could be added to each batch were made on a record sheet carried on each transit mixer, the driver-operator making a corresponding record of the quantity of water actually used. The time of beginning of mixing and the time of completed discharge were also noted on the mixer record sheet so that a complete record was obtained for each batch. The clock type indicating water meter was set for each batch so that it automatically shut off the water and clearly indicated the amount used. A master accumulative dial was read and noted on the mixer record at the beginning and end of each shift to indicate the total quantity of water used, to serve as a check against the quantities shown for the individual batches. The daily transit mixer records were filed in the field laboratory and were valuable in showing the water required for the desired consistency and also the time required to batch, mix and deliver a mixer load of concrete. The time interval between the start of mixing and placement of the concrete was not in excess of 30 minutes for the longest haul of five miles, while for short hauls of about one-half mile this time interval was approximately 10 minutes. The average for the whole job was about 20 minutes.

The concrete was delivered by the transit mixers to the canal lining machine which placed, vibrated, and formed it in position. This machine consisted of a steel plate shaped to the canal section and attached to a double stee truss which spanned the canal and was supported on a track on each side by two-wheeled trucks. A structural steel frame was erected above the deck of the machine to support two I-beams which acted as overhead tracks for the concrete buckets. From each overhead track a bottom dump bucket with a capacity of two-thirds of a cubic yard of concrete was supported by a trolley which rode on the lower flange of the I-beam. Concrete from the transit mixers was discharged through a metal chute into the buckets, which were pushed by hand along the rails and discharged through a wide slot in the deck of the lining machine. The overhead tracks were built so that the buckets would pass, and one could be filled while the other was being

discharged. The design of the transit mixers was such that the concrete was discharged at a high level, which aided materially.

The transit mixers also effected economies in the construction of the structures by the elimination of stockpiling with attendant waste of aggregates and expensive hand batching. Much closer control of the concrete was also possible with the central batching plant than would have been possible if the aggregates had been stockpiled along the canal.

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Job Problems and Practice

WHAT JPP MEANS

—an informal clearing house for mutual help on concrete problems; an opportunity for the ACI member who *knows* to tell the ACI member who *needs* the information. If the subject matter holds probabilities of general interest it will be briefed in these pages—with authorship credit unless the contributor prefers not.

If you know of an interesting problem whose solution has smoothed the way of someone in the field, record it briefly. If you have a problem, present it; a question, ask it. If you are moved to helpful comment or constructive criticism write the Secretary.

The "answers" are those of individuals to whom the questions are referred and not of the ACI.

Fabricating and Erecting Column Forms (38-81)

BY CHARLES MACKLIN†

Here are several methods of building column forms. All of them require accuracy and considerable work in the office, on the bench, and in erection. Possibly the most common method is to fabricate the columns squaring the bottom and letting the tops extend above the cut off point. This works, but it has several disadvantages, namely, the concrete on which the column rests is not necessarily level and the job of getting cut-off grades is hampered by obstructions or excessively long sights, not to mention the awkward position of the carpenter who must do the cutting.

A procedure which eliminates the objectionable features has been used with success; accompanying figures will help to explain it in detail. The forms are fabricated from the beam bottom down and are stacked near the bench until a bottom cut-off is determined. For convenience a mark can be made on the column at the time of its fabrication. This mark can be any convenient distance above finish floor, preferably about

†Normandy, Mo.

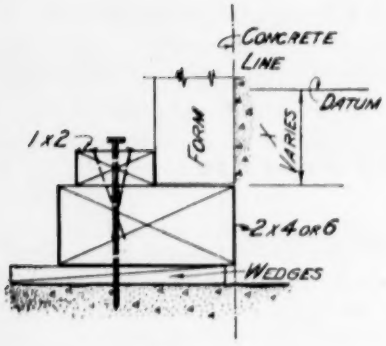

Fig. 1

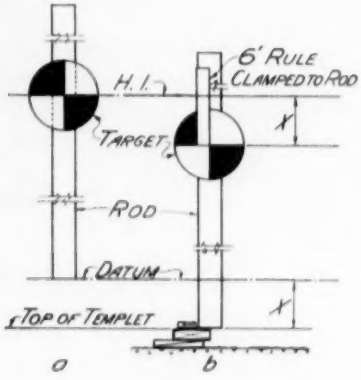

Fig. 2

6 in. above the rough concrete. In Fig. 1 and 2 this elevation is referred to as datum.

Rarely are columns placed without some sort of templet to locate and hold the columns. This templet can be made as shown in Fig. 1 and has the added advantage of being level. If it is spiked to the concrete before it gets too hard no other method need be resorted to in holding it. The secret in spiking wood to concrete is in not driving the head of the spike home so the head will compress the wood and pull the spike loose.

When a number of these templets are in place the engineer can get the dimension "X" (Fig. 2) quickly. If the target is set so that the bottom of the rod is on datum, then when the rod is held on the template the target is X distance below HI. By holding or clamping a rule to the target this dimension may be read directly in inches. Noting the column number and this dimension on a piece of paper with a carbon and contact with a field book a record can be retained and the piece of paper can go to the bench. At the bench the carpenter makes the necessary cut-offs and marks the columns accordingly.

Effect of Aluminum Powder on Durability of Concrete (38-79)

Two questions—on the use of "Aluminum powder to increase bond of concrete to reinforcing steel" 38-78 (answered by F. R. McMillan) and the "Effect of aluminum powder on durability of concrete" (38-79), answered by Roy W. Carlson, were published in the November 1941

JOURNAL, p. 183-184. Professor Carlson now supplements his reply with the following:

Later tests using somewhat more aluminum powder have shown very large improvements in the ability of concrete to resist freezing and thawing, especially where thawing is with calcium chloride. The most striking results are obtained by first intergrinding very fine aluminum with sand and adding enough to cause several per cent of voids.

Research Man's Troubles* (38-82)

We have underway some tests on freezing and thawing which will give us a comparison of the behavior of regular portland cement, moderate-heat and low-heat cements. These tests have been going on for more than a year, but it would be very unwise to make a report on them. We found some very decided changes in the behavior of some of the specimens over a period of a few weeks and, therefore, we are withholding all information until most of the specimens have failed.

It has been our practice to review test data on research problems in concrete and endeavor to interpret trends at six-month periods when the investigation extends over two or three years. This sometimes causes a lot of explaining when the data go into reverse and we are convinced that a researcher in concrete should wait until all of his test specimens are destroyed and then be certain that all materials are scrapped and no more available, and then he can draw his conclusions and interpret the data without danger of having to withdraw any of his statements.

Some tests on drilled cores from structures gave such erratic results that I am not willing to publish them. There seems to be no rime nor reason about the behavior of the concrete taken from structures, although at one time we thought we have something very splendid in the relationship between the results obtained by freezing and thawing tests and the ratio of fine aggregate to total aggregates. The trouble is we extended the tests too far and included cores from some other structures and thereby upset a beautiful theory.

How to Prevent Efflorescence (38-83)

Following the 1941 convention presentation of the paper by Messrs. Hutcheroft and Anderson, "The Use of Carbon Dioxide to Reduce Efflorescence on Asbestos-Cement Shingles"†. H. F. Thomson, St. Louis, asked this question: "We hear a great deal about the possible

*By a well-known research man who prefers to be anonymous.

†Clyde R. Hutcheroft and Harvey R. Anderson, ACI Journal, Apr. 1941; *Proceedings* V. 37, p. 597.

use of puzzolanic materials in the concrete mix as an agent to react with free lime and thereby prevent efflorescence. Can it be used to advantage?" Mr. Hutchcroft's response: "We have tried puzzolanic materials. Shingles are applied when they are 30 to 60 days old, and puzzolanic materials do not react fast enough with calcium hydroxide to provide a solution to the problem. There is as much bloom with the addition of puzzolanic material as there is without it."

Some Details in Designing and Mixing Concrete (38-84)

BY H. G. COLLINS*

The subject of mix design recently has occupied considerable space in the JOURNAL. The purpose of this contribution is to discuss a few details which have proven helpful to us and may interest others. Two of these points apply in particular to the trial batch method, while the third is a method of handling the aggregate.

Let us review very briefly the steps of the trial batch method. Starting with a specified water cement ratio or cement factor, an estimate is made of the water requirement per cu. yd., basing it on whatever knowledge is available regarding the aggregates to be used. With the amount of cement and water fixed, the absolute volume of the cement paste is computed, and from this follows the absolute volume of aggregate. The proportions of fine and coarse aggregate are estimated from the gradation, and the absolute volumes are converted to weights from the specific gravities. This procedure was described by Stanton Walker in the JOURNAL for January, 1939†.

Mixing a trial batch of the ingredients as indicated by those computations serves two main purposes, first to correct the balance between fine and coarse aggregate by adding one or the other as seems desirable from the texture of the concrete, and second to use just enough water to give the desired consistency, regardless of the amount estimated in advance. Thus, concrete is mixed to approximate what is desired, and the problem now becomes one of using the data to make a final design.

From the volume of the trial batch, the number of such batches required to make one cubic yard is computed, and this multiplied by the weight of each ingredient in the trial batch gives the weight of each per cubic yard. If the consistency was just right, the water requirement per cubic yard is known, otherwise a small correction can be made. As a rule of thumb, a 3 per cent change in water can be used for a 1-in. change in slump. The cement will require adjustment to obtain either

*Lehigh Portland Cement Co., Allentown, Pa.

†JPP (35-21) *Proceedings* V. 35, p. 211.

a fixed cement factor or a definite water-cement ratio, but the water requirement will not be affected by such a change.

The volume of the trial batch might be computed as the sum of the solid volumes of the ingredients, but the result is subject to error due to possible inaccuracies in the specific gravities and to the air content of the mix. A better method is to fill and weigh a measure of concrete. The batch weight divided by the unit weight of the concrete gives the volume of the batch independently of specific gravities and the unknown amount of air in the mix.

In the adjustment of the quantities computed from the trial batch to give the final design, we come to our first point. The obvious method is that which was used to design the trial batch, outlined in the second paragraph, using the corrected quantities of cement and water. We find it more dependable, however, to make the final design by adding to or subtracting from the quantities of the trial batch. The absolute volume of the necessary changes in cement and water is compensated by an equal absolute volume change in the aggregate. To illustrate, assume that the design calls for 6 sacks or 564 lb. of cement, but the trial batch contained 543, requiring an addition of 21 lb. Assume also that the water should be reduced from 324 to 320 lb. to make the mix slightly stiffer. The absolute volume of the change in cement is +.107 cu. ft., in water −.064, a net change of +.043. This is compensated by subtracting an equal volume from the aggregate, the weight of which would be $.043 \times 2.60 \times 62.4 = 7$ lb. The changes are small, hence inaccuracy in specific gravities has a negligible effect, and the air content of the mix is not ignored.

Obviously, it is important that the filling of the measure with concrete be accurate, and that is the second point. Our method of accomplishing this is to use a glass plate, as was described by J. C. Pearson in the JOURNAL for June 1939*. The measure is over-filled slightly, the concrete is trowelled smooth, and a glass plate is slid over the top with a sawing motion to cut off the small excess.

As an indication of the dependability of the procedure described, Table 1 gives results obtained recently on 12 cements covering a wide range in characteristics. A single trial batch was made for each mix, and the results shown are the averages of three measurements.

The third point concerns that bugbear, "a saturated, surface-dry condition of aggregate." Air-dry aggregate is more convenient to handle. Suitable corrections in the calculations can be made if the absorption from air-dry to saturated surface-dry is determined, and the air-dry aggregate will change little if stored in closed containers. However, when consistency is involved, the aggregate should be saturated

*JPP (35-21); *Proceedings* V. 35, 0, 584.

CEMENT CONTENT AND SLUMP OF 12 MIXES

Designed for	Cement, sks. per yd., 4½	Slump, Inches 2½	Cement, sks. per yd., 6	Slump, Inches 2½
Cement No.	Results Obtained on Final Batches			
1	4.50	3.25	5.97	2.3
2	4.50	3.2	6.00	2.6
3	4.51	3.0	6.01	2.6
4	4.50	2.5	6.00	2.4
5	4.50	2.75	6.02	2.3
6	4.50	2.2	6.00	2.9
7	4.49	3.0	6.00	2.4
8			6.00	3.0
9			6.01	2.6
10			5.99	2.5
11			6.00	3.0
12			6.00	2.6

Slumps expressed to tenths of an inch because each is the average of three.

when used. It seems a simple solution to the problem to moisten the aggregate in advance of use with a known amount of water—more than will be absorbed, but somewhat less than will be required for the batch. A special pan has proved very helpful in doing this. This pan is 6½ in. deep, 16 in. x 12½ in. on top, 10 in. x 11½ in. on bottom, and has a loose-fitting, slip-over cover with a drop edge of 1 in. The air-dry aggregate is weighed and then moistened in the pan, after which it is placed in the fog room for an overnight period, at least. There is no evaporation, so the loose cover is good enough, and the drop edge prevents any drip from entering.

The sloping ends are a convenience in emptying the aggregate into the mixer or mixing pan, and in rinsing. A battery filler (a syringe having a rubber bulb and hard rubber stem) is used to wash out the grains of sand that stick to the pan. With judicious use of water, 200 cc. will be enough, and this, also, is from a known amount. The pan should be dampened before use. In this way, very few grains of sand or drops of water are lost.

The flare of the sides and ends allows them to be nested together when not in use. Our only difficulty with them is that they are too handy for other uses.

"Pops" in Slag Concrete (38-85)

Q—There are records of "pops" appearing on exposed surfaces of concrete in which slag had been used as the large aggregate. Will you explain the cause of the phenomenon?

BY FRED HUBBARD*

A—In discussing this question of "pops" which, on very rare occasions, have been known to appear in the surface of slag concrete, I am confident that I am going to be a great disappointment to you when I say that we don't know a great deal about the more subtle phases of the incidents mentioned, which is largely accounted for by the fact that we have had such limited experience with their occurrence.

In the thousands of structures and the hundreds of miles of highways built with slag concrete we have recorded less than 20 cases where popping occurred, some of these being the appearance of only a single spall in a project containing several thousands of yards of concrete. We have studied each case very closely and a survey of the slag industry has been accomplished to determine if the trouble-makers could conceivably be developed in sufficient quantity to become potentially dangerous from a structural standpoint, or whether their appearance was merely irritating as an easily corrected disfigurement of the surface of the concrete involved. The matter was finally placed in the latter category.

Whoever posed this question also desires, I assume, to know how the material that pops in slag concrete can be developed, how often it will make its appearance, number of cases that have been brought to my attention, and, in general, a description of the seriousness of the whole affair.

In answering these details it will have to be assumed that most of you are somewhat familiar with the process of smelting iron ore in a blast furnace. In brief, a large steel stack which is lined with refractory materials is the chief piece of apparatus. In the stack are assembled, in precisely determined amounts and time intervals, iron ore, fuel and a fluxing agent. The ignited fuel, aided by a heavy air blast, builds up very high temperatures (ranging from 2500 to 3000 F.) within the stack and a continuous chemical process ensues as successive portions of the "burden" are renewed.

When the metallic iron is freed from the original ore, the remaining compounds, silica and alumina, combine with the lime and magnesia of the fluxing agent and the solids remaining from the combustion of the fuel and form slag.

In spite of the intense heat of the furnace it is possible, under very rare freak operating conditions, for fragments of the flux to pass down from the unburned material above and be drawn off with the molten slag at the time the furnace is being tapped. These fragments are usually abnormally high in silica and magnesia. Almost invariably they are

*Director of Research, National Slag Assn., at ACI "Quiz" Session, 37th Annual Convention, Washington, D. C., February, 1941.

rendered functionless when they have reached atmospheric temperature as they will take up enough moisture to break down into a harmless powder during the cooling process.

When, however, some few of these pieces have clung tenaciously to their trouble-making properties, their presence is announced by the appearance on the exposed surfaces of the finished concrete of a single example or a collection of cone-shaped "pops" or "pop-outs". These cones vary in depth, generally from a half inch to an inch and a half. A very few have been found that reached the extreme depth of two and a half inches. They arrive with no regularity of pattern, number or size in any noted "case". The material that has caused the trouble is clearly discernible after the action has occurred and, in turn, their size varies from that of a match head to as large as a walnut.

Now, to answer as to the extent of the disturbance caused, and the frequency of their occurrence, we will have to step into the realm of pure conjecture because, as far as I know, there is in existence no formula that would enable anyone to compute the number of square feet of exposed surface of the approximately forty million cubic yards of slag concrete that has been placed in the last 20 years.

But if we were to ignore the total square footage of all of the exposed slag concrete in all of the highway and railroad bridges, the docks, the mills, the factories, and the warehouses, and approach the subject from the slag concrete highway angle only, I may be able to satisfy you.

The estimated 500 miles of 20-foot wide slag concrete highways that have been built since our attention was called to the first case of "popping", some 20 years ago, would have exposed 52,800,000 sq. ft. of surface. In the "pop census" we have accumulated in that time there have been less than a total of 1,000 "pop-outs", and the average of the area of the bases of the cones produced was something under one-quarter of a square foot—let's say their total was 250 sq. ft.

One can now establish a ratio that would be conservatively expressed in the following equation:—

$$250 \text{ sq. ft.} : 52{,}800{,}000 \text{ sq. ft.} = 1 : 211{,}200$$

250 divided by 20 equals 12½ sq. ft. annually, or an area approximately the size of the top of an ordinary office desk.

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Saving Steel in Reinforced Concrete Design*

BY R. L. BERTIN†
Member American Concrete Institute

SYNOPSIS

The author proposes the prompt modification of the Institute's Building Regulations for Reinforced Concrete (ACI 318-41) in furtherance and support of a proposed National Emergency Code for the conservation of steel and other strategic materials, chiefly for defense construction where time is urgent and steel short; that working stresses be liberalized, live loads reduced, plain concrete encouraged and the most modern methods of design applied.

The nature of most structures to be erected during the period of emergency with which we are faced will of necessity be of a temporary nature to fulfill some very specific function linked with the emergency. They will be needed as fast as they can be built with little time available for considering their fitness in the scheme of things when normalcy returns.

Their permanence, convertibility, etc., must be subordinated to the all important considerations of speed of erection and minimum consumption of critical materials. These things must govern in their planning and construction and normal methods of procedure and concept of design must be revised to satisfy the new demand.

Under ordinary conditions, the selection of the type of structural material used is generally guided by (1) The kind of service the structure is to render; (2) The initial relative costs of equally satisfactory materials, and (3) The personal preference of the designer.

Under existing conditions, the designer, in selecting a structural material must be guided by its availability and utilize the critical materials for want of a substitute and in the most efficient manner—the relative cost being secondary.

*Received by the Institute, Jan. 2, 1942.
†Chief Engineer, White Construction Co., New York, N. Y.

Steel conservation dictates that aside from structures for which timber is adequate, most of them be built of reinforced concrete rather than structural steel, wherever possible.

Most existing codes regulating structural design were not written with the idea of meeting the present emergency and the various code writing agencies should revise their regulations, giving recognition to the most recent concept of design with a view of making available steel go as far as possible.

As members of the Engineers Defense Board's sub-committee on steel conservation, J. P. H. Perry† and the writer were called upon to submit their recommendations for drafting an emergency building manual intended to supersede the existing codes for the period of the emergency.

AN EMERGENCY CODE

The report, the text of which follows, was submitted to and approved by the Board and incorporated in the recommendations which it prepared at the request of the O. P. M. on the subject of Steel Conservation and Reclamation.

Basically, the members of this Sub-Committee fully agree that an all inclusive building manual liberalized to the extent of permitting types of construction and materials, working stresses and live loads which would reduce to a safe minimum the amount of steel used in buildings, could effect a very material reduction in the quantity of steel used.

This belief is based on the facts that the existing building codes controlling the great majority of construction work throughout the country have not kept pace with the development of the art and that an ultra conservative tendency has guided the drafting of those which have been modernized.

In ordinary times this policy is justified, but in the case of a national emergency the principles of economics, the resulting instinct guiding the men responsible for the planning of commodities, do not apply, for thrift is no longer measured in terms of dollars, but in terms of materials.

In order to utilize materials to their utmost, it is necessary that the ones responsible for their use in kind and quantity be given a free hand. The multiplicity of codes scattered over this land more or less restrictive in nature, seriously handicaps designers in utilizing available materials to their utmost capacity. To overcome this conditions, either the code making bodies must revise their codes or the existing codes must be stayed and superseded by an emergency manual prepared by a central agency.

The former is a hopeless undertaking, therefore the latter appears to be the only logical solution.

Mechanics of producing a national emergency code

It is evident that such a manual should be completed within the shortest possible time. The committee responsible for it should be composed of representatives of national organizations such as the American Standards Association, governmental

†Turner Construction Co., New York, N. Y.

departments, particularly the Army and Navy, steel, concrete, and lumber institutes, representatives of engineering societies and the fire underwriters.

In general, this manual should be made applicable to all structures needed for the national emergency, bearing in mind that many will be of a temporary nature.

Benefits of such code

In order to accomplish the purpose of this code, namely to conserve critical materials, it should:

(1) Liberalize working stresses.

(2) Reduce live loads for which structures are proportioned to actual safe maximum conditions.

(3) Encourage the use of plain masonry wherever possible, as in footings, retaining walls and the like.

(4) Liberalize the use of wood for the construction of roofs and roof structures.

(5) Discourage, and in some cases, prohibit the use of steel for the construction of items where less critical materials, such as wood, plastics, concrete, and gypsum can be used instead, for instance:
   (a) Steel pipe piles and sheet piling
   (b) Steel floors
   (c) Window sash and doors
   (d) Partitions
   (e) Steel guards at platform edges and column corners, railings and fencing
   (f) Sheet metal flashings
   (g) Marquise
   (h) Flag Poles
   (i) Metal furniture

particularly when the construction is of a non-permanent nature.

(6) Liberalize the plumbing, electrical and fire protection regulations.

(7) Revise zoning restrictions, if necessary, in order that they operate within the intent of this proposed emergency period manual.

(8) Encourage the development of concrete road construction using reduced steel reinforcement.

How to make code effective

In order to make such a manual effective, it should be distributed by the O. P. M. to
   (a) The City officials of every city in the United States having a population of 10,000 or more.
   (b) All State and Federal agencies concerned with construction.
   (c) All licensed architects and engineers.
   (d) All members of engineering societies.
   (e) Such other groups as are directly or otherwise connected with construction, such as engineering schools and colleges, money lending institutions, etc.

General remarks

In the draft of this manual, the most recently adopted codes should be secured and reviewed with a view of adopting the desirable features found therein.

Such provisions as those stipulated for the construction of the temporary buildings at the New York World's Fair should be given serious consideration.

Its effect on steel conservation would be expedited by promulgating the sections progressively starting with the most important ones such as "Loads and Stresses" and structural design.

During the life of this manual, i.e. during the period of National emergency, a standing committee should be active in keeping the document up to date and in re-writing sections which may appear incongruous or subject to later improvement.

SUGGESTED REVISIONS OF ACI CODE

The items of the report falling within the scope of the ACI are listed below and are the subject matters discussed in this address:

(1) Liberalizing working stresses.
(2) Reduction of live loads.
(3) Encouraging the use of plain concrete.
(4) Application of most modern methods of design.

The most effective way of reducing the amount of steel reinforcement used in flexural reinforced concrete members is to make available bars of high yield point and stress them at a higher rate than is now the custom.

Tests conducted by Lyse[1], Mylrea[2], Johnston[3], Krefeld[4], and others, have demonstrated that in reinforced concrete beams, reinforced with high yield point steel the full resistance of the bars can be developed fully.

The inclusion in the ACI Code of the regulation permitting a unit tensile stress of 50 per cent of the yield point, but not in excess of 30,000 psi on $\frac{3}{8}$ ϕ bars used in one way slabs, the span of which does not exceed 12 ft. was based on the results of some of these tests[1][2].

The more recent New York City code revision, permitting a unit stress on high yield point bars not to exceed 40 per cent of the yield point or 24,000 psi, followed the satisfactory results of beam tests which served to reaffirm the generally accepted fact that within limits the ultimate strength of a reinforced concrete beam is proportional to the yield point of the steel used to reinforce it.

European countries, in order to conserve steel, have for some years past resorted to the use of high yield point bars stressed at much higher rates than is permitted in this country.

In view of the fact that the present National emergency has brought about a shortage of steel, it is highly desirable that the production of high yield point steel bars be encouraged and that the use of correspondingly higher unit stresses be sanctioned by authoritative organizations. It is therefore urgently suggested that the Institute take steps to endorse such action by modifying accordingly the present section of its code dealing with steel stress.

The utmost saving of steel in the construction of reinforced concrete columns demands that they be proportioned, with no more reinforcing steel than required to insure their lateral stability and resistance to flexure when tension is involved.

While for heavy loads such columns will be larger, column sizes can be kept down through the use of high strength concrete and the structures planned to accommodate the larger columns if necessary. The use of spirals as well as high percentages of vertical steel should be discouraged.

In this connection, it seems desirable that the Institute reconsider the column section of its Code with a view of increasing the permissible unit stress on rodded columns and reducing the required minimum amount of vertical steel below the present 1 per cent limitation. It is suggested that the requirements of the New York City Code relating to tied columns be given full consideration. It is worthy of note that a great many columns which have been designed and constructed in conformity with these regulations have given and are giving satisfactory service.

Reduction of live loads

The widely varying live load requirements for buildings found in the building codes throughout the country, indicate the necessity for a standardization. While this phase of structural design does not come within the scope of the Institute, nevertheless it should receive its attention since it affects the factor of safety. Bearing in mind the temporary nature of most structures, the live loads for which they are proportioned should not be any more than their specific requirements indicate.

Encouraging the use of plain concrete

Certain parts of structures now built of reinforced concrete because of the economies resulting therefrom, such as piles, footings, retaining walls and the like, should wherever possible be designed of plain masonry.

Probably because of the fact that the use of plain concrete structures has been supplanted almost entirely by that of reinforced concrete, the development of design method for its application has received scant attention in the field of research. The ACI code although defined (Sec. 101) as applying to plain and reinforced concrete contains no provision for the design of unreinforced concrete structural members.

An appreciable amount of steel can be saved by encouragiing the use of plain concrete footings, gravity retaining walls, concrete cast in place, piles in which neither permanent steel shells nor reinforcement is used, concrete roads designed to reduce the reinforcement to a minimum, and the like. Accordingly, it is suggested that the Institute promulgate rules for the application of such structures.

In summing up, the conservation of steel in reinforced concrete design resolves itself into a few simple rules: (1) Let the concrete do all the work it is capable of doing; (2) Such reinforcement as is used should be

high yield point steel, and (3) Liberalize the working stresses to an extent consistent with a safe but not excessive factor of safety.

The indications point to the creation of an agency within a very short time for the purpose of drafting a building manual for use by Governmental bureaus, State and City departments, etc., in liberalizing their codes with a view of reducing the quantity of steel used in structures coming under their respective jurisdictions.

The American Concrete Institute can render a most important service in leading the way toward a safe and effective reduction of steel consumption in the application of reinforced concrete to structures. It is urged that its various committees be called upon to reconsider the regulations for which they are responsible from the standpoint of the present emergency and modify them to the fullest extent consistent with safety in order to conserve steel. Such modified regulations endorsed by the Institute would be of invaluable assistance to such agencies as will be called upon to liberalize existing regulations.

In planning reinforced concrete structures, the designer should have in mind the relation which the quantity of steel used bears to the spacing of supports, the load for which it is proportioned, the depth of the flexural members, the unit stresses to which the materials are subjected and the shape of the structure. In general, this relation is expressed by the basic qualitative equation

$$Q = \frac{W.L.}{R.D.S.}$$

in which W = Load
L = Span
R = Structural Shape factors
D = Depth of member
S = Allowable working stress
Q = Quantity of steel

In addition, the flexural members should be proportioned so as to keep the shear stresses below the point where large quantities of stirrups or bent bars are required, and the bond stress within the upper limit set for ordinary anchorage, thus saving the steel required for hooks and extra lengths of anchorage.

Little can be said regarding the relative steel consuming characteristics of the various types of floor and roof construction, such as beam and girder, two way slab, flat slab, arch roof, shell roofs, etc. The experience and judgment of the designer, backed by comparative designs is to be relied upon in reducing the amount of steel used in a given structure.

In closing, let it be said that in the final analysis making all those involved in the design and construction of structures Steel Conservation conscious is the prime requisite and it is hoped that this paper will be effective to that end.

REFERENCES

(1) Inge Lyse and George R. Wernisch, "A study of reinforcement in concrete slabs," ACI JOURNAL, Sept.-Oct., 1936; *Proceedings* V. 33, p. 1.

(2) T. D. Mylrea, "Concrete slabs reinforced with welded wire fabric," ACI JOURNAL Nov.-Dec. 1935, *Proceedings* V. 32, p. 219; and "Effect of impact on reinforced concrete beams," ACI JOURNAL, June, 1940, *Proceedings* V. 36, p. 581.

(3) Bruce Johnston and Kenneth C. Cox, "High yield-point steel as tension reinforcement in beams," ACI JOURNAL, Sept. 1939, *Proceedings* V. 36, p. 65.

(4) W. J. Krefeld, "Comparative tests of concrete beams reinforced with Isteg and hot rolled deformed bars," Columbia University Report No. 2507, April 1941.

Discussion of this paper should reach the ACI Secretary in triplicate by June 1, 1942, for publication in the JOURNAL Supplement, concluding Proceedings V. 38.

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Discussion of a paper by R. L. Bertin:

Saving Steel in Reinforced Concrete Design*

CONVENTION DISCUSSION BY FRANK E. RICHART, CHARLES S. WHITNEY, DEAN PEABODY, JR., HERMAN SCHORER, THOR GERMUNDSSON, J. E. KALINKA, MORTON O. WITHEY, LEO KAUF, GEO. S. STREHAN, CHESTER L. POST, W. C. SPIKER, HALE SUTHERLAND and BRUCE JOHNSTON, CARLETON S. PROCTOR, LANE KNIGHT; Since Convention by CLYDE T. MORRIS.

By FRANK E. RICHART†
(presiding officer)

Mr. Bertin seems to think that ACI should take some action on the recommendations in his paper. I believe it proper for me to say that, so far as our Building Code is concerned, we have no power to act here in revision of a Standard of our Institute, unless the Board of Direction should change the whole machinery of our procedure.

By CHARLES S. WHITNEY‡

The writer endorses Mr. Bertin's suggestion that a manual to supercede existing codes be prepared for the design of emergency structures. Increased efficiency in the use of materials and labor is necessary at this time and it will be equally important during the period of reconstruction after the war. Many current practices which are needlessly wasteful should be corrected as quickly as possible.

The problem involves not only the correction of code provisions which are too conservative, but what may be more difficult, the teaching of modern methods of design to men who have had years of practice with the older methods. The issuance of a liberal emergency code, properly emphasizing progressive methods would be extremely helpful in overcoming the inertia of old practices. It should permit the use of

*ACI JOURNAL, Feb. 1942; *Proc.* V. 38, p. 281. The announced schedule called for closure on the discussion of Mr. Bertin's paper June 1 and publication in the *Supplement* next November.
The timeliness of the subject and the many contributions to the discussion at the 38th Annual Convention, Chicago, Feb. 20 prompt the publication here of such discussion available as this April JOURNAL goes to the printer. Discussion is still open until June 1.

†Research Professor of Engineering Materials, University of Illinois, Urbana.
‡Consulting Engineer, Milwaukee, Wisconsin; Chairman ACI Committee 312, Plain and Reinforced Concrete Arches.

new methods of design or construction which research has shown to be rational. Competent designers would then be freed from the restriction of code provisions which have not recognized recent technical developments.

The design of even temporary emergency structures must be safe and in accordance with sound engineering principles. If there is any possibility that the use of a structure may be extended beyond the period of the emergency its parts should be designed for an economic life consistent with the type of material used so that the investment will not be lost because of structural weaknesses.

Where reinforced concrete structures can be satisfactorily used instead of structural steel a saving in steel will be effected. The desire to make a further saving in the reinforcing of concrete should not be permitted to result in uneconomical structures as long as sufficient steel is available. In a controlled market, however, it may be difficult to decide how much increase in total cost will be justified by a given saving in steel. If the true value of the steel could be estimated this would be a simple calculation. In using plain concrete instead of reinforced concrete, care should be taken to avoid waste of labor and materials not warranted by the saving in steel.

In line with the four general methods for saving steel, which Mr. Bertin has suggested, the writer makes the following specific recommendations:

(1) Revision of allowable steel stresses;

(2) Recognition of actual strength of concrete and reinforcing steel in compression:

(3) Adoption of rational method of design for members under bending and direct load.

(1) Steel stress

As stated by Mr. Bertin, it has been proven that up to the limit of balanced reinforcement, the flexural strength of a beam is proportional to the yield point strength of the steel bars. According to the Joint Committee Report the allowable stress, the yield point stress and the consequent factor of safety for the three ordinary grades of steel are as in Table A:

TABLE A

Grade of Steel	Minimum Yield Point psi	Allowable Tensile Stress psi	Factor of Safety
Structural	33000	18000	1.83
Intermediate	40000	20000	2.00
Hard	50000	20000	2.50

It is evidently inconsistent to use a different factor of safety for each grade of steel. Intermediate grade bars now obtainable actually have a minimum yield strength of 45,000 psi and it appears that the specification could be changed to that figure without any difficulty. A similar change could probably be made in the yield strength of structural grade bars, but that may not be important because they are not often used. For emergency construction a factor of safety of two for the tensile steel should be sufficient and for many structures a maximum design stress of 25,000 psi would not cause excessive cracking in service. Following this rule the specification would be as in Table B:

TABLE B

Grade of Steel	Minimum Yield Point psi	Allowable Tensile Stress psi	Factor of Safety
Structural	36000	18000	2
Intermediate	45000	22000	2
Hard	50000	25000	2

(2) Compression strength of concrete and steel

It has further been demonstrated* that the ultimate resisting moment of a beam with balanced reinforcement as limited by the concrete strength is equal to

$$M = \frac{bd^2 f'_c}{3}$$

With a factor of safety of 2½ the allowable load could be calculated from the equation

$$\frac{M}{bd^2} = \frac{f'_c}{7.5}$$

According to the standard straight-line formula, the allowable value (*) of $\frac{M}{bd^2}$ varies with the steel stress although tests show that the ultimate strength of a beam in case of compression failure is independent of the tensile steel stress. For five different steel stresses and a 3000 lb. concrete Table C gives the allowable $\frac{M}{bd^2}$ and the corresponding factor of safety.

There seems to be no justification for this increase in factor of safety as the steel stress is increased, and it will offset to some extent the advantage of using higher steel stresses. A factor of safety of 2½ against

*"Plastic Theory of Reinforced Concrete Design" by Charles S. Whitney, *Proceedings*, Am. Soc. C. E., Dec. 1940.

TABLE C

Steel Stress psi	Allowable $\frac{M}{bd^2}$	Factor of safety against concrete compression failure
16000	262	3.82
18000	248	4.03
20000	236	4.23
22000	224	4.46
24000	214	4.67

concrete failure should be sufficient if it is assumed that it makes no allowance for variation in concrete quality. Separate consideration should be given to the value of f'_c to assure that it represents a safe minimum value. The factor of safety used in design should not be complicated by the concrete quality factor which may vary so greatly from job to job. It is absurd to penalize all jobs with a high factor of safety regardless of how well the concrete may be controlled. The proper method would be to make an estimate of what value of f'_c can safely be used in each case depending on job conditions.

The general use of a working value of $\frac{M}{bd^2}$ as high as $\frac{f'_c}{7.5}$ will not be economical because of the large amount of steel required but it will often be desirable because it will save concrete and compression steel at points of high bending. It permits the designer a greater leeway in making an economical design.

When more steel is used than provided for balanced reinforcement by present standard codes, the plastic theory formulas presented by the writer* should be used because the standard straight-line formula gives too high a value of the steel lever arm and would result in a reduction of the factor of safety against steel failure. Below this range the writer's formula gives a somewhat greater steel lever arm than the standard formula and saves some steel. Both of these variations from the standard formula are fully supported by tests.

Important reduction in the amount of compression steel required in beams, and frames can be made by proper recognition of the true strength of the concrete in bending. Further saving can be made by recognition of the fact that if buckling and bond failure are avoided, compression steel in beams will develop its full yield strength before failure. The working stress could therefore be the same as that for tension steel. The present Joint Committee provision that the effectiveness of compression steel in beams "may be taken at twice the value indicated from the calcula-

*See previous reference.

tions, assuming a straight-line relation between stress and strain" is a poor attempt to compromise with the facts. Even so, no corresponding allowance is made for columns under highly eccentric loads in which case the compression stress is limited by the straight-line formula to n times the concrete stress adjacent to the steel.

(3) Design of eccentrically loaded columns

A great saving can be made by proper revision of the method of designing eccentrically loaded columns which is at present very inadequate. The method given in the Joint Committee Report provides an

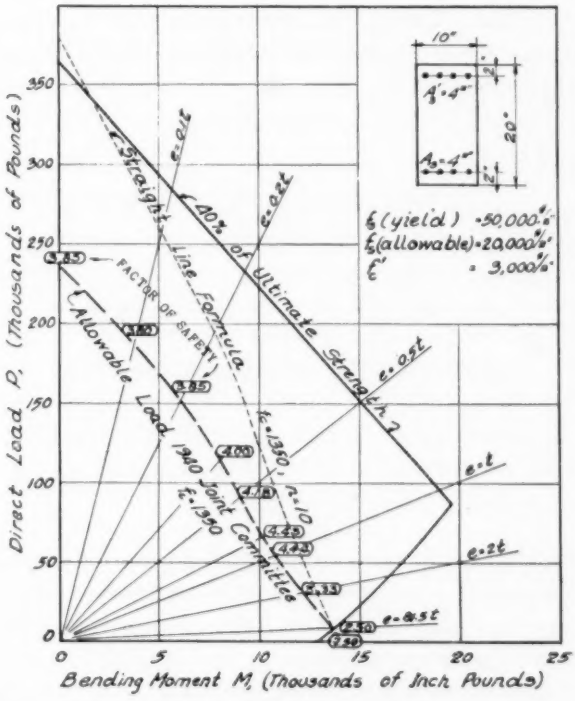

Fig. A—Comparison of allowable load with ultimate strength of eccentrically loaded columns

inconsistent factor of safety and is so indirect that it discourages designers from making complete calculations. The increasing application of frame analysis to concrete structures, makes this important.

Committee 312 has made an exhaustive study of members under combined bending and direct load and has proposed a simplified and rational method of design based on tests*. (See Fig. A.) The chart

*"Plain and Reinforced Concrete Arches" Report of Committee 312, Charles S. Whitney, Chairman; ACI JOURNAL, Sept. 1940.

included herewith gives for a particular member a comparison between the actual strength and the allowable load according to the Joint Committee. The chart is plotted with the direct compression as the ordinate and the bending moment as the abscissa. The radial lines are lines of equal eccentricity. It is possible in this way to plot in one quadrant the performance of a member under the full range of eccentricity from zero to infinity (axial load to pure flexure).

The heavy solid line represents forty per cent of the ultimate strength for each eccentricity and provides a factor of safety of 2½. The dash line shows the Joint Committee allowable load. The factor of safety provided by the latter varies from 3.85 for axial load to 4.44 when $e = t$ and then down to 2.5 when the eccentricity is very great. This value of 2.5 is due to the use of hard grade steel. With intermediate grade it would be 2.

Because a so-called axially loaded column will be subjected to accidental eccentricities a liberal factor of safety is proper, but it is evident that for calculated eccentricities the column has much more safe strength than can be utilized under the Joint Committee Report.

The writer suggests that the formulas proposed by Committee 312 be used for the design of eccentrically loaded columns. An application of these formulas to round columns has also been made. The safe load for axially loaded columns revised as suggested by Mr. Bertin should be used as a maximum for small eccentricity.

The application of the recommendations of Committee 312 will permit the economical use of long span concrete arch construction instead of structural steel for such buildings as hangars with the added advantage of increased fire resistance. Adequate consideration has not yet been given to the possibilities in that direction.

In preparing an emergency code, it should be recognized that the present emergency will probably last several years. Even if the war does not last long, a period of reconstruction must follow in which efficiency in the use of materials will be necessary. Although the simplest method of saving steel is to increase working stresses, the benefits will be limited by the inaccuracy of the straight-line theory. Even if it cannot be used immediately by all designers because of their unfamiliarity with it, the plastic theory should be made available for use so that design practice may be put on a more progressive basis.

By DEAN PEABODY, JR*

This short paper is both timely and disturbing. It is timely since the course of events forces attempts to save steel, and will lead to an increase of reinforced concrete construction. It is disturbing, because one realizes that the emergency dictates immediate action without time for a careful survey of the effects of changes in code requirements. These changes, made necessarily in haste, will probably continue in force after the emergency is over, much as the increased shear stresses for reinforced concrete ship construction in the last war were continued under "special anchorage" after the war ended.

It will be a distinct advantage if the emergency results in the adoption of a single code for reinforced concrete design throughout the country, instead of the multitude of city codes differing often only in details.

If economy is secondary and the saving of steel is of first importance in design, immediate savings can be made without code changes by the use of plain concrete footings, gravity retaining walls, etc.

The tied column of high strength concrete reinforced with a low per cent of longitudinal steel has often been more economical in the past than the smaller column with a greater per cent of steel, even if allowance be made for the extra floor area occupied. This is especially true for axial loads. The strength of the concrete is of less economic importance for columns subjected to bending. The tied column can be kept within normal size limits for structures six to eight stories high. Higher buildings, or heavier floor loads than usual, will run up the column size to four or five feet thick, if spiral steel or structural steel columns are ruled out.

Increase in size of members means increase of dead weight and load on the supporting member. These increases are not important as far as footings and columns are concerned, but a general increase in size of floor systems would be a serious matter. Saving of steel by refinements in design and change of code requirements will be of the greatest value in slab and beam designing.

The assumption of straight line concrete stress variation in general use for beam design has given economical designs for concrete strengths of 2500 psi, or thereabouts, with comparatively low percents of steel. An increase in the allowable steel stress, using the same concrete strengths, will result in a larger beam and greater dead weight, but with less total steel. However, the use of high strength concrete and high yield point steel will give a smaller beam section and a reduction in total steel, while the shear stresses can also be kept within ordinary anchorage

*Associate Professor of Structural Design, Mass. Institute of Technology, Cambridge, Mass.

requirements. The tests quoted seem to justify use of concretes of 4000 psi strength. The straight line theory employs the modular ratio, $n = \frac{E_s}{E_c}$ where $E_c = 1000\ f'_c$. It is recognized that the modulus of elasticity of concrete E_c is greater than this for elastic deformations due to load changes, but this low modulus is justified if allowance is to be made for the effects of shrinkage and plastic flow. It would seem wise not to change the modular ratio radically.

There has been increased interest in the method of beam design for ultimate loads presented by Charles S. Whitney.* Compared with the straight line theory, the economical section for the same stress requirements is smaller but the area of steel is greater, amounting to 2 or 3 per cent. This, of course, is opposed to the emergency requirement of reduction in steel. If the steel area be limited to that given by the straight line theory, the beam section is larger and the dead weight is increased.

A code change allowing a unit steel stress of 40 per cent of the yield point for high yield point bars will give a definite saving of steel and a reduction of dead weight for beams made of high strength concrete. An upper limit for concrete strength ought to be set in order not to extrapolate too far beyond the available test data.

By HERMAN SCHORER†

The author has excellently stated the steel design factors by means of his basic qualitative equation (p. 286). The working stresses and loads are evidently the most important items affecting the quantity of steel. A consistent consideration of these two factors must necessarily lead to the possibilities of prestressed concrete construction. Although such methods are still in the development stage, the following remarks may be of some interest.

Liberalizing working stresses

As pointed out, higher steel working stresses in flexural members would most effectively reduce the amount of steel reinforcement. To a lesser extent the amount of steel is also influenced by the concrete stresses in shear, bond and compression. Nevertheless, the practical limitation of all working stresses is indirectly determined by the low tensile strength of all concrete. It is well known that the assumed monolithic action

Charles S. Whitney: "Design of Reinforced Concrete Members, under Flexure or Combined Flexure and Direct Compression", ACI JOURNAL, March-April 1937, *Proc.* V. 33, p. 483; Kenneth C. Cox: "Tests of Reinforced Concrete Beams with Recommendations for Attaining Balanced Design", ACI JOURNAL, Sept. 1941, *Proc.* V. 38, p. 65.

†Borsari Tank Corporation of America, New York, N. Y.

between steel and concrete will be endangered, if excessive cracks are permitted in the principal tension zones.

Under emergency conditions the permissible width of the cracks may be liberalized, depending on the purpose, load conditions and expected life of the structure. For this reason the production of high yield point steel bars appears justified, although the amount of steel thus saved is difficult to estimate.

The wire industry already produces commercial spring wire, music wire, piano wire, etc., with a yield point range from about 120,000 to almost 300,000 psi, depending on the size and quality of the wire. Ordinary reinforced concrete design and construction are not adapted to utilize a 50 per cent yield point stress of this steel, since the structure may crack wide open and even fail under its own weight, long before the wire stress has reached safe limits.

Reduction of live loads

The Abrams water-cement ratio law has brought about a considerable improvement in the compressive strength of concrete. Nevertheless, most structures must still be designed to carry largely their own weight, instead of useful live loads, similar to the load proportions of standard railroad equipment.

A less conservative live load estimate will certainly reduce the amount of steel, but a substantial dead load reduction would be much more effective, provided this can be obtained with available materials and within safe limits. Furthermore, any favorable shift in the live load-dead load proportion would widen the application of concrete for long span construction, thereby releasing structural steel for more urgent needs.

A radical improvement in the prevailing load proportions can hardly be expected, as long as the width of the cracks, rather than the yield point of the steel and the compressive strength of the concrete control ultimate design limits.

Prestressed concrete

Prestressed concrete construction attempts to eliminate principal concrete tensile stresses under all normal load conditions. The designer is then in a position to take advantage of the better compressive strength of modern concrete and the excellent tensile properties of cold drawn steel wire.

Such tests were made by Lund and Koenen.* More recently, Freyssinet† has made considerable progress. Finsterwalder and Dischinger‡

*"Der Eisenbetonbau", by E. Mörsch, Vol. 1, first half, p. 371.
†"Spannbeton nach den Freyssinet Verfahren", by K. W. Mautner, *Beton und Eisen*, 1936, p. 320.
‡"Neues Bauen in Eisenbeton", *Deutscher Beton-Verein*, Berlin, 1938, pp. 18 and 24.

and others have proposed special methods, which are not of the truly monolithic type of construction.

Hoyer* has shown that highly stressed, relatively thin wires are self-anchoring, when embedded in concrete under initial tension. Hoyer's tests were made with hot drawn, lead patented steel wire (about .04 to .12-in. diameter), with a reported tensile strength of 340,000 to 383,000 psi and a 90 per cent yield point. Precast beams and slabs of the Hoyer type are now extensively fabricated in Germany to replace lumber with so-called "steel-poor" reinforced concrete, in accordance with tentative code revisions.

The monolithic prestress methods as used in Europe are practically limited to precast members. They would require a prohibitive amount of equipment and labor, if applied for more general construction purposes.

The main difficulty is found in the temporary accommodation of the large steel tension force reactions. There can be no advantage in reducing the quantity of reinforcing steel if, at the same time, a multiple amount of structural steel is required for the construction of unyielding anchorages or extremely heavy steel forms.

Fig. B—Ten ton capacity pre-stress unit 25 ft. long bent under stress

A more general approach to this problem was found in the conception of self-contained reinforcing units (Fig. B) which are preloaded and placed into ordinary wooden forms, similar to reinforcing bars. The numerous wires and their initial tensioning forces can thus be handled in convenient bundles, like packaged goods, adapted for standardized production in different sizes and preload capacities.

In the present case, the stretching force reactions are temporarily transmitted to a corresponding high yield point compression member, contained within the bundle. After embedding the preloaded unit in concrete and upon proper hardening of the same, the compression member is released by means of the protruding end connection (Fig. C). The wire tension reaction is thereby transmitted to the concrete in form of initial, internal compression. Both end connections and the compression member are removed for reuse in the fabrication process.

*"Der Stahlsaitenbeton" by Ewald Hoyer, published by Otto Elsner Verlagsgesellschaft, Berlin, 1939.

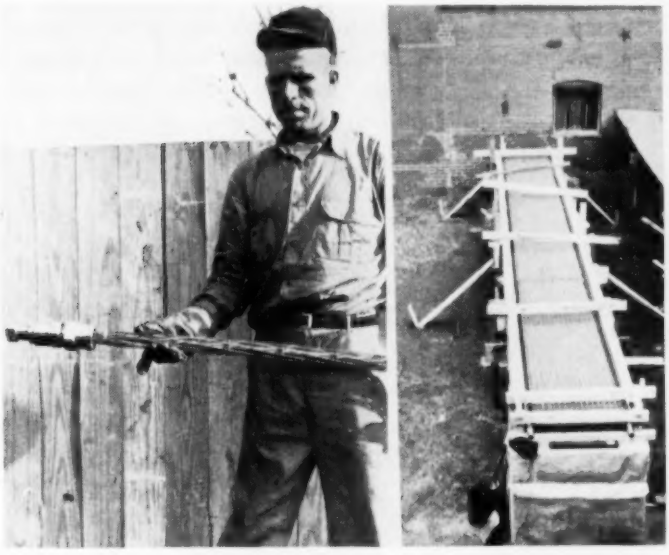

Fig. C (left)—Prestress unit end connection
Fig. D (right)—End connections protrude from each rib

It will be noted from Fig. B that the prestress units are easily curved under stress within given limits. This feature is of advantage in beam or girder construction, and may be compared to the purpose of bent-up bars. The so-called diagonal tension near the supports is thus effectively reduced so that shear stresses are of minor importance.

An experimental program has been carried out in St. Louis since 1939. A test roof slab (Fig. D), with span of 28 ft. 6 in. between supports, has ribs 10¼ in. high; thus the height-span ratio is about 1:33. This slab is designed for a live load of 30 lb. per sq. ft. (live load-dead load ratio 1:1), and prestressed in the longitudinal direction by two units of 10 tons capacity each, with an initial steel stress of 125,000 psi. The observed dead load deflection is minus ¼ in. A uniform live load of about 40 lb. per sq. ft. gave a deflection of plus .35". Up to this load the deflections are proportional to the load and no hair cracks were detected.

Further progress has now been interrupted pending availability of the negligible amount of material and equipment required for experimental purposes. Practical applications will become feasible as soon as the necessary facilities for the fabrication of prestress units can be made available. Preliminary estimates for simple structures indicate possible steel savings of about 60 to 80 per cent compared with ordinary designs.

The extent and duration of the present emergency cannot be foreseen. It is quite possible that existing and now entirely obsolete steel structures may be considered as a source of scrap steel. Shall they be replaced with wood, which is neither fireproof, splinterproof nor plentiful? Concrete certainly would be much more desirable for various reasons.

The writer fully agrees with the author in letting the concrete do all the work it is capable of doing and to use high yield point steel. It seems desirable that the suggested revisions of the ACI code include a provision for prestressed concrete design, based on actual test results. In contrast to many emergency compromises, any beginning made in this direction will gradually advance and widen the field of reinforced concrete construction.

By THOR GERMUNDSSON*

Mr. Bertin said ". . . that little can be said regarding the relative steel consuming characteristics of the various types of floor and roof construction". I have some data on this subject which may be of interest.

Comparison of concrete floor systems

In 1940, before the steel shortage became acute, seven types of concrete floor systems were designed including the supporting beams.† The studies included three live loads—50, 100, 150 psf, and three span lengths—15, 20, 25 ft.

Comparative cost was then the object, but today, while cost is still a vital problem, it is the steel quantities that are of special interest. The following steel quantities in lb. per sq. ft. of floor area for seven types of floors are taken from these design studies for 20-ft. span and 100-lb. loading.

1. 20-in. metal pans	2.77
2. 30-in. metal pans	2.58
3. 12-in. masonry filler	3.54
4. 16-in. masonry filler	3.43
5. One-way slab	3.01
6. Two-way slab	3.70
7 Flat slab	2.13

One point that stands out is the superiority of the flat slab design, which requires only 2.13 lb. of steel per sq. ft.‡ It not only uses considerably less steel than the next lowest type, which requires 2.58 psf, but as shown in the original studies, the flat slab is also the most economical floor design system.

*Portland Cement Association, Chicago.

†By the Structural Bureau of the Portland Cement Assn. Design based on ACI Code 1936 with f_s = 20,000 psi, f'_c = 2,500 psi.

‡Flat slab here means girderless solid slab with drop panels, column capitals and two-way reinforcement.

The designs that have most steel are types 3, 4 and 6—floors with masonry fillers and the two-way solid slab. The floors with metal pans and the one-way slab are between the extremes.

Type 5, the one-way solid slab, deserves special comment. The steel factor of 3.01 psf is based upon a tensile stress of 20,000 psi but this stress may be increased to 25,000 according to the Joint Committee Report of 1940* for bars not exceeding ½ in. and to 30,000 according to the American Concrete Institute Code 1941 for bars not exceeding ⅜ in., provided the stress does not exceed 50 per cent of the yield point stress. Since small size bars are readily obtainable with a minimum yield point stress of 50,000 psi, the slab bars may be designed for 25,000, which means that 20 per cent of their weight may be saved. Making the correction in type 5 brings the steel factor, including bars for supporting beams, down to 2.53 psf. The one-way slab then moves into the second-best position.

Neither flat slab or solid slab ceilings need metal lath or plaster. Ceilings that require suspended ceilings with metal lath should have approximately one-half pound added to their steel factors. Types 1 and 2, with suspended ceiling included, will then require approximately 3.2 psf, and compared with this figure both flat slab and solid slab show a definite margin of saving.

Flat slab construction has been regarded as suitable especially for heavy warehouse loads. Actually, flat slab will often show a saving both in cost and steel for light load construction such as apartments; and it deserves consideration in many other occupancies.

Weights of steel per sq. ft. of floor area taken from the studies are plotted in Fig. E. Increasing the load increases all steel factors, but the smallest increase in steel is in the flat slab, the superiority of which becomes more marked the greater the load.

Comparison of reinforcement in columns

The saving of steel in reinforced concrete columns will be illustrated by reference to a number of designs for which steel weights are given in Table D. The column load assumed is 500 kips, and the steel weights are given per column with 12-ft. story height. Various assumptions and design conditions are given in the table. Four design codes are included in the study.

Comparison of items 1 and 2—Both are designed with minimum steel, but item 1 has ties while 2 has spirals. The tied columns require much less steel but they are larger than the spiral columns. It is surprising

*Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete.

TABLE D

Col. load 500 k. A structural steel column of equal capacity would weigh 1300 lb. per 12 ft. story height.	f'_c = 3000										f'_c = 5000						f'_c = 3750			
	1		2		3		4		5		6		7		8		9		10	
	Minimum steel				Same size as 2		Size reduced compared with 2, 3				Min. steel		Size reduced compared with 6				Size reduced			
	Tied		Spiral		Tied		Tied		Spiral		Tied		Tied		Spiral		Tied		Spiral	
	Col. size	Reinf. lb.	Col. size	Reinf. lb.	Col. size	Reinf. lb.	Col. size	Reinf. lb.	Col. size	Reinf. lb.	Col. size	Reinf. lb.	Col. size	Reinf. lb.	Col. size	Reinf. lb.	Col. size	Reinf. lb.	Col. size	Reinf. lb.
J.C. 1940	28	387	25	630	25	643	22	937	22	788	22	257	20	545	20	653	22	681	22	630
A.C.I. 1936	30	444	25	630	25	944	—	—	22	812	22	574	—	—	20	677	22	984	22	658
N. Y. 1938†	26	174	25	448*	25	212	22	916	22	723	—	—	—	—	—	—	—	—	—	—
Chi. 1937	26	347	25	517*	25	464	22	782	22	695	22	249	20	374	20	536	22	507	22	569

†f'_c = 3400 is max. allowed. *These values are close to min. steel.

NOTE.—Weight of reinforcement is in lb. per 12 ft. story height. 24 dia. added for lapped splices. 15 lb. added for ties per column. 15 lb. added for spiral spacers. Spirals assumed 11 ft. high. Hot rolled spirals. All columns are square.

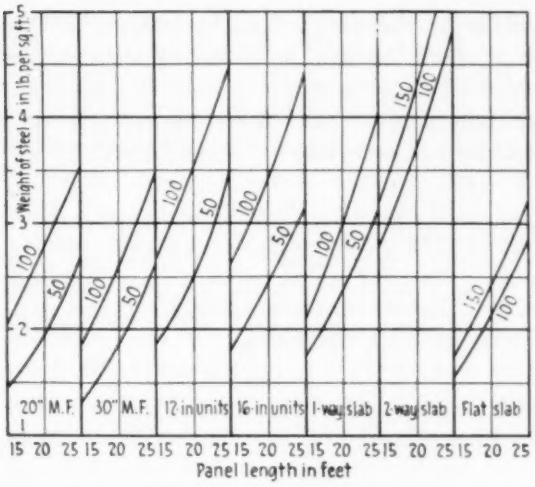

Fig. E—Weight of steel in lb. per sq. ft. based on one exterior slab panel plus two-thirds of one interior beam. Panels are square with dimensions between 15 and 25 ft., loads between 50 and 150 p.s.f. Ceiling lath is not included

though that only a one-inch difference in size is required by both New York Code 1938 and Chicago Code 1937. Yet the reduction in steel for tied compared with spiral columns is 61 per cent for New York 1938 and 33 per cent for Chicago 1937. Under the conditions in items 1 and 2, it is evidently advantageous to use tied columns and, if there is a choice among the codes, to use New York 1938, for which the steel weight is only 174 lb. per story. A structural steel column of equal capacity would weight 1300 lb.

Comparison of items 2 and 3—All the columns are 25 in. square, but item 2 has spirals while 3 has ties. Tied columns designed by ACI Code 1936 have abnormally high steel factors throughout. This is because in a last minute change on the convention floor and against the advice of the committee which prepared the code, a load factor was changed by vote from 0.8 to 0.7. These columns will be disregarded in the following, since the column formulas in the 1941 edition of the ACI Code are the same as in the 1940 Joint Committee Report.

Comparing 2 and 3, it is seen that the tied column has the same steel (J. C. 1940) or much less steel (N. Y. 1938) compared with the spiral column. Note particularly New York 1938 for which steel in the tied column is 53 per cent less than in the spiral column. Yet the size is the same. Again we see an illustration of the superiority of the tied column and of the New York Code in respect to column design.

Comparison of items 4 and 5.—All the column sizes are reduced to 22 in. square, and for such relatively small sizes, the tied columns require more steel, but not a great deal more than the spiral columns. The condition is reversed where the design is based upon 5,000-lb. concrete as will be discussed later.

Comparison of items 4 and 6—All the sizes are 22 in. square, but the concrete strength is 3,000 in 4 and 5,000 in 6. By raising the concrete strength we have reduced the steel 73 per cent for J. C. 1940 and 68 per cent for Chicago 1937. The conclusion is that when loads are large and column sizes are small, a great deal of steel may be saved by using high-strength concrete.

Comparison of items 7 and 8—9 and 10—Items 7 and 8 are 20-in. columns with 5,000-lb. concrete. It is rather surprising to see that the tied columns in 7 have less steel than the spiral columns in 8. Items 9 and 10 are 22-in. columns with 3,750-lb. concrete, and under these conditions tied and spiral columns have approximately the same steel requirements, ignoring the ACI Code 1936. In other words, relatively small column sizes with high-strength concrete will often require less steel when ties are used instead of spirals. On the whole, tied columns appear from the studies to be superior to spiral columns in regard to conservation of steel.

Comparison of building frames

The question naturally arises as to how much steel can be conserved by converting a building frame from an all-steel to a reinforced concrete structure. In ordinary times, when steel is plentiful, a comparison resolves itself into a question of cost. But today the viewpoint has changed, and attention is being focused on steel consumption.

In 1940, five already erected buildings with steel frame were redesigned in reinforced concrete complete with floor framing plans and column schedules. No special attempt was made in any of the designs to conserve steel since the sole object then was one of cost. Yet, information taken from the bills of materials prepared will serve to illustrate the question now under consideration.

The five buildings varied so widely in respect to location, height, occupancy and layout that the results obtained are believed to be representative of a great variety of multistory building frames. Table E shows weights of steel per sq. ft. of floor area for both types of frame and also the amount of steel saved by the reinforced concrete design.

TABLE E—STEEL PER SQ. FT. OF FLOOR AREA

Project No.	I	II	III	IV	V	Ave.
S/S—lb/sq. ft.	17.44	18.29	15.21	15.01	9.19	15.03
R/C—lb/sq. ft.	6.21	6.87	6.40	5.93	3.09	5.70
Saving—lb/sq. ft.	11.23	11.42	8.81	9.08	6.10	9.33
Saving—per cent	64	62	58	60	66	62

Although the weights of steel vary considerably, the saving in per cent is remarkably uniform. It varies from 58 to 66 per cent, the average being 62 per cent. The saving in cost by using concrete was much less uniform. It varied from 23 to 59 cents per sq. ft., with an average of 52 cents per sq. ft., which was one-third of the average cost of the structural steel frames including floors and ceiling treatment.

The foregoing conclusions apply to multistory buildings which have relatively few or no exceptionally long spans. One-story structures with long-span roof construction will be discussed in the section that follows.

Long-span roof construction

Data on steel consumption for multi-story building frames showed an average saving of 62 per cent due to the use of reinforced concrete. A similar saving is computed for a one-story industrial building with relatively long spans and bays in the roof construction.

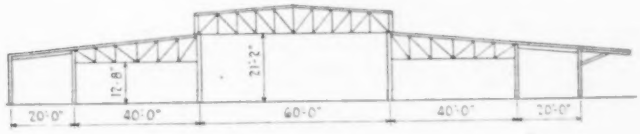

Fig. F

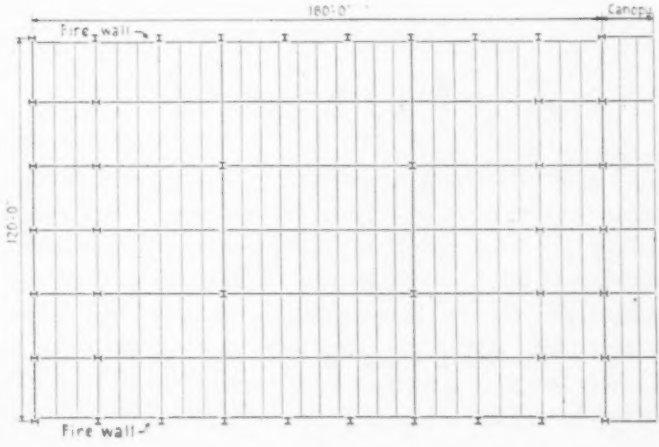

Fig. F shows the roof framing for a warehouse of a type used in the present war period. There are five buildings, each 182 by 1,582 ft., out to out, plus a 16-ft. canopy. The transverse spans are 20-40-60-40-20-ft., except in the firewalls where they are all 20 ft., and the longitudinal column spacings are 20 and 40 ft. The three interior spans have light

trusses, and the entire roof has 2-in. wood decking. The structure is not fire-resistive, but concrete firewalls divide the building into lengths of 120 ft. The design is typical of many one-story manufacturing and storage buildings being constructed at present, in which much steel is being wasted. The particular buildings under discussion were not built according to this design, but steel tonnage was estimated as 7,120 for five buildings.

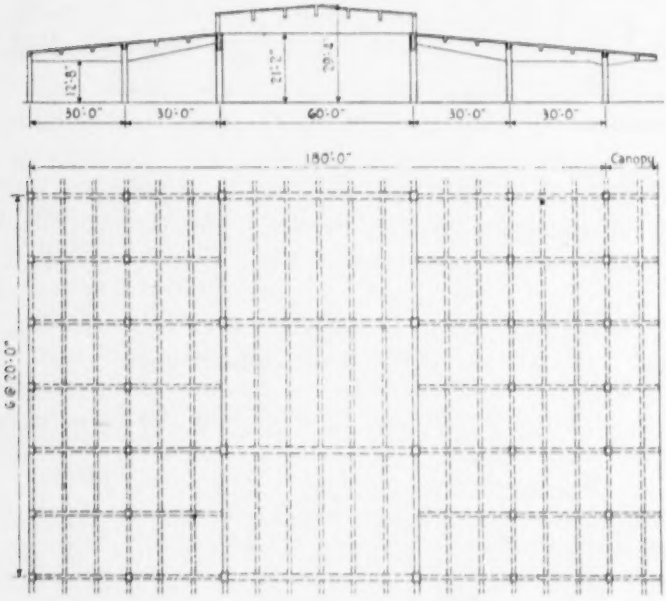

Fig. G

The framing in Fig. G illustrates a redesign in reinforced concrete. The two outer spans that were 20 and 40 ft. in the original layout are changed to 30 and 30 ft. More significant, the spacing of the columns in the firewalls which was 20 ft. is changed to 30-30-60-30-30 as in the other concrete bents. The longer spans in the firewalls are less favorable to the concrete design, but it is better to have all bents alike in case the firewalls some day may have to be removed to make room for some other kind of occupancy.

Despite this unfavorable change, the concrete redesign requires only 2,367 tons of reinforcing steel. The saving in steel, 4,753 tons or 6.2 psf, amounts to 67 per cent of the tonnage in the all-steel design. It must be said, however, that the saving is probably large because the concrete structure was designed taking into account many of the means of saving steel discussed in these pages.

TABLE F

	1—Present practice Design load: DL & LL				2—Recommended practice Design load: ¾ DL & LL				3—Ideal case Design load: ½ DL & 1½ LL			
No.	I	II	III	IV	I	II	III	IV	I	II	III	IV
DL/LL:	0.5	1.0	2.0	4.0	0.5	1.0	2.0	4.0	0.5	1.0	2.0	4.0
DL	6,700	10,000	13,300	16,000	7,300	11,400	16,000	20,000	5,700	10,000	16,000	22,900
DL + LL	20,000	20,000	20,000	20,000	21,800	22,800	24,000	25,000	17,100	20,000	24,000	28,600
DL + 2LL	33,300	30,000	26,700	24,000	36,300	34,300	32,000	30,000	28,600	30,000	32,000	34,300
DL + 3LL	47,700	40,000	33,300	28,000	50,800	45,800	40,000	35,000	40,000	40,000	40,000	40,000
DL + 4LL	60,000	50,000	40,000	32,000	65,300	57,200	48,000	40,000				
DL + 5LL	73,300	60,000	46,700	36,000								
DL + 6LL	86,700	70,000	53,300	40,000								

Consistent method of increasing allowable stresses

Table F contains three divisions. No. 1 represents present practice, No. 3 is an ideal case, and No. 2 illustrates an intermediate condition. Each covers four ratios of DL/LL. Various loading cases are considered —including one dead load plus a certain number of live loads, the number varying from zero to six.

The stresses in No. 1 are computed for given loads and ratios in a structure designed as in present practice on basis of DL+LL and a constant working stress of 20,000 psi as indicated in the second line of stresses. It is seen that ultimate stress, 40,000 psi, is reached under 2.5LL when DL/LL = 0.5 but under 6.0LL when DL/LL = 4.0. This is just one way of demonstrating again the fact long recognized, that the present manner of treating factor of safety is grossly inconsistent. The St. Louis Code has tried to improve this situation by allowing design to be based on DL+4LL and a constant ultimate stress, but this proposal has not been taken up by other codes.

No. 3 illustrates what happens if we design for DL+3LL and a constant ultimate stress of 40,000 psi. It is an idealized method of eliminating the inconsistency prevailing in No. 1. Note particularly the stresses under the loading of DL+LL. They vary from 17,100 to 28,600 psi. The point is that if the stress is kept constant for, say, DL+3LL, it cannot also remain constant for DL+LL. Therefore, if we want to improve present practice, we must operate with working stresses that have different values for different load ratios.

This is where the difficulty lies. Everybody agrees on the principle involved but nobody likes to design for variable working stresses. The principle was embodied in the 1931 edition of the A.A.S.H.O. bridge specifications but abandoned in subsequent issues. The practical obstacles were considered too great.

We are fortunate in having a very simple and ingenious way of overcoming also the practical obstacles. One proposal is studied in No. 2 in which we design for a load of ¾DL+LL and a constant working stress of 20,000 psi. Let us investigate stresses for the various loadings and ratios noted in the table. The results are interesting.

First, observe the stresses for DL+LL in No. 2. They are variable as they should be, but the designer need not know it, or bother about it, because it comes out that way automatically when he designs for ¾DL+LL and a constant working stress of 20,000. All the designer has to do is to multiply all dead loads by ¾. Live loads are in this case left unchanged, and everything else is the same as in present practice.

By simply multiplying all dead loads by ¾, which actually means that the dead load factor of safety has been reduced by 25 per cent, we

have accomplished several things. First, we have raised the general level of allowable stresses above the current value of 20,000, which is considered desirable to conserve steel during the war. Second, we have taken a long step toward establishment of factors of safety that are consistent for all ratios of DL/LL. Third, since we have not gone as far as in No. 3, we still retain the advantage of having greater margin of safety for greater ratios of DL/LL. Finally, it seems a reasonable and consistent way to conserve steel by increasing all stresses but to raise them most where it is needed most.

Whichever way we look at it, the new type of design load has advantages. It is also highly adjustable. As new conditions arise, we may alter the load factors at will, but still keep on designing for the same constant working stress. The design load is the only thing that changes. It is hoped that this simple procedure will remove all practical objections to a use of factors of safety which is more consistent than in present practice.

By J. E. KALINKA*

Mr. Bertin's paper is timely and worthwhile. Recognition of the necessity to reduce consumption of critical materials and to consider the availability of certain types, shapes, etc. of such materials was the forethought of at least some, the more progressive, government engineering agencies in advising some of their consulting engineers to disregard prevailing codes, even those issued by these agencies, and to carry out the most economical design in accord with the best knowledge of the art. I think this in itself was a definite step forward, to appeal to the skill of the designer, yet a safe step because no conscientious engineer would jeopardize his reputation by going beyond the limit of safety as established by his best understanding.

An "Emergency Code" is desirable, if not written in a strict code form, but rather compiled as a guide, giving certain directions and limitations.

Liberalization of working stresses will be an effective means of conserving critical materials, but it can also be a dangerous step if arbitrarily applied. I call your attention to a very worthwhile discussion on that subject by Bruce Johnston,† who comments on the proposed increase of allowable unit stresses for steel structures. I quote:

> Even if the buckling and fatigue problems are avoided, increases in allowable unit stresses should imply a shift from a factor of ignorance to a load factor or true factor of safety in structural design. Hence, the engineer who uses such increased stresses should always ask himself the question: "Has the factor of ignorance decreased in inverse ratio to the increase in design unit stresses?"

*Roberts & Schaefer Co., Engineers, Chicago.
†*Engineering News-Record*, Feb. 12, 1942, p. 87.

* * * while we know a lot more today about *how* to analyze stresses in structures it does not follow that this knowledge is usually put into practice. More often than not it is entirely impractical, if not impossible, actually to make a complete stress analysis of a structure. The important thing has been *to build and use* a structure, not to analyze it.

The simple, approximate formulas for average or so-called maximum stresses which are used today do not tell anything about the localized maximum stresses in fillets, etc.

Design accuracy must compensate for increased stresses

The thoughts are to some extent applicable to reinforced concrete design in that, if you increase stresses, you have to "make up" by greater design accuracy and by considering design assumptions beyond those usually taken into consideration. For instance, if a design takes into account not merely dead and live loads but also influences due to volume changes of concrete, possible predetermined unequal settlement of supports, etc., it is definitely justifiable to increase stress limits similarly, if not necessarily in the same measure, as conceded for wind and earthquake loads. It must also be remembered that in larger spanning structures possible excessive deformations might restrict the use of higher unit stresses.

On the other hand there are a number of conditions which, quite generally, permit a liberalizing of working stresses as, for instance, those cited by Bertin. I fully agree on the increase of steel stresses in slabs of small spans, on the reduction of the minimum percentage of steel in stubby reinforced concrete columns, etc. I also want to add that a saving in diagonal shear reinforcing can be obtained by using smaller and more closely spaced reinforcing bars, which is in agreement with the findings expressed in design codes abroad permitting, for closely spaced diagonal tensile reinforcing, an increase in stresses up to 100 per cent above those permitted in connection with the use of heavy and widely spaced bars. Reinforcing in wall panels can be saved if more frequent expansion joints are provided, or if the edges of these wall panels are more heavily reinforced, permitting a considerable decrease of reinforcing in the center portion of wall panels. In certain cases reinforcing steel can also be saved by resorting to the design of "wall-like" girders (diaphragms) as self-carrying units in which there is a propitiously large effective depth.

A very important point mentioned by Bertin is to encourage the use of plain concrete, and his words should be heeded: "Let the concrete do all the work it is capable of doing". Let me tell you a story of an experience some 15 years past. My company had designed and built a great number of underground pits and hoppers for railroads averaging 20 x 20 x 20 ft.; some of these were subject not only to earth pressure and the dynamic forces imposed by railroad tracks above, but even to

hydrostatic pressure. In making a rough analysis I was first baffled in as far as a considerable amount of reinforcing bars seemed to be required in the 12-in. and 18-in. walls and floors, if then prevailing building codes had been followed. The fact that these structures stood up for years and in fact still today give service and are water-tight led me to a more thoughtful analysis of these structures by considering the closed frame action—both horizontally and vertically—with the result that if such structures are analyzed as they actually behave, i.e., as three dimensional boxes, the maximum stresses, both compressive and tensile in the plain concrete walls, amounted approximately to but 200 to 250 psi. Here, manifestly, the concrete had done and still does all the work.

It seems that such striking behavior should tend to utilize plain concrete more frequently in structures not subject to changing atmospheric conditions and excessive volume changes in concrete. In arguing the factor of safety of such plain concrete structure one has to consider that the above-mentioned 250 psi maximum tensile fiber stress is arrived at by straight line theory. The actual concrete tensile stresses, by virtue of a curvilinear distribution of fiber stresses within the bent section, may be only 50 per cent or 60 per cent of the theoretical value or 125 to 150 psi. This brings me to the point I want to make, i.e., that saving in reinforcing steel can also be obtained in certain types of structures and structural members if the tensile resistance of concrete is taken into account.

Although not directly connected with the interest of the forum, it is mentioned in Bertin's paper that we liberalize the use of wood for roof structures, which also applies to wider spanning form work for concrete structures. In this connection I call your attention to glued laminated wooden arches, frames and beams which came on the market during the last few years. These products were rigidly tested by the U. S. Forest Products Laboratory.* They consist of laminae scientifically arranged, pressure-glue-welded, into solid members. By virtue of carefully selecting the material and arranging laminae so that the grain follows one principal stress plane, it is logical that higher unit stresses than those now permissible for natural timber could be used.

As mentioned in the last paragraph of Bertin's paper, "making all those involved in the design and construction of structures steel conservation conscious is the prime requisite". However, it is just as important that those involved in the negotiation of contracts be even more conscious of the possibilities for steel conservation, and it is hoped that the necessary cooperation may be accomplished by making an "Emergency Code" available to those responsible all down the line.

*U. S. Department of Agriculture Bulletin No. 691.

By M. O. WITHEY*

The liberalization of working stresses has been mentioned as one of the possible means of lessening the use of steel. It seems to the speaker that for more important structures, working stresses already appear sufficiently high for beams and slabs. Resistance to corrosion and water tightness demand that cracking be reduced to a minimum. Increasing working stresses will cause wider cracks and more deflection of slabs and beams, regardless of the yield point of the steel used. In columns the use of low percentages of deformed high elastic limit bars and high strength concrete is a possibility worthy of consideration. Attention should also be given to the possible use of high strength plain concrete with about a half to one per cent of spiral. Steel can be conserved by making beams and slabs deep and columns larger. The use of the light weight aggregate will afford some reduction in dead load. Wherever information exists concerning the character of the live load, full advantage of that information should be taken into account in designing. The use of plain concrete in footings should be permitted only when the stresses in the footings are those permissible for plain concrete. In general it does not appear wise to increase working stresses for important structures, but it does appear wise to change methods of design and use sizes of members not now considered efficient under normal conditions. The advantages of pre-cast and pre-stressed members should receive thorough consideration. The possible use of Bessemer steel and other steel not satisfactory for services demanding the highest grade of steel, should be carefully considered for the manufacture of reinforcing bars and mesh.

By LEO KAUF†

I want to refer to the explanation given by Mr. Schorer of the saving of steel, through pre-stressing. There are two schools on steel savings; one plan is to take high yield point steel and stress it to over twenty, thirty or forty thousand pounds per square inch, without regard to cracks in the tension zone. The other school is to take tension altogether out of the concrete by pre-stressing. These two methods have been combined by a method, first evolved by Emperger, which is being extensively discussed in London just now, where refugees from France and from Central Europe are leading a very heated battle. The school of combining the two methods is afraid of pre-stressing alone, because they claim that if the pre-stressing of concrete is done without sufficient supervision, the fate of a pre-stressed structure is dependent on the conscientiousness of the man who pre-stresses. Well, Mr. Schorer has shown us a very ingenious device by which this factor is mitigated, but this

*Professor of Mechanics, University of Wisconsin, Madison.
†Member I. Struc. E., London, England, present at ACI convention.

device naturally has not been known to those who use this argument. These people propose stressing high-yield-point steel up to forty and fifty thousand pounds per square inch, and pre-stressing the tension zone to minimize the cracking in the concrete. The combination has not been used extensively, and we have no data as to results, while we know the results of pre-stressing alone. We know the work of Freyssinet in bridges, and the replacing of normal steel joists by precast prestressed joists. For these, the wire is stretched in one length of several hundred feet, and goes through the single forms laid out in one row end to end. After the concrete has hardened, the wires are cut. This is a very inexpensive method of pre-stressing, and should be taken up extensively.

Another subject I should like to mention is in connection with research undertaken by myself in designing wide-span precast construction. In 1940 I had to design, upon the request of the British Government's Building Research Station, an arched-roof of a hundred and sixty foot span, without using any lumber, and very little steel. The previous roofs were constructed of 650 tons of structural steel. The final concrete design required one hundred tons of cold developed reinforcing steel.

After the events in Norway, there was no lumber available for concrete forms; pre-cast concrete was the only solution. Preliminary tests had to be made, as there is no literature available indicating how to join axially pre-cast units and take care in joining of bending moments and shear. The tests consisted in loading beams of eight foot span to failure, and doing the same with two pieces of four feet each, joined together by different methods, thus ascertaining the loss of moment of resistance. It proved advisable, in the end, to use twin units and stagger the joints, because the spread in strength, through overlapping the steel for joining, was as much as 25 per cent, according to the conscientiousness or negligence of the worker.

A recent article on this subject* dealt also with the possibility of using reinforced concrete for one-story structures.

By GEO. E. STREHAN†

In the emergency war effort, while our energy must be applied to immediate expansion and new construction for defense production, a word for cautious procedure is pertinent. Just as there exists an all-out present emergency, there will also be a post-war period of reconstruction. To the extent that normal public and private improvements are now held to an absolute minimum, much of the new construction will find

*Leo Knuf, *Concrete*, Oct. 1941.
†Consulting Engineer, New York, N. Y.

a place in post-war development. Wherever possible, emergency developments should be directed along lines of permanent usefulness, or recognition be given to purely temporary character in the design.

War-time activity has already manifested itself in extensive population shifts accompanied by demands for war-time housing and human habitation facilities as well as industrial construction. In 233 cities with populations varying from more than 500,000 to less than 25,000 recent population shifts have necessitated rezoning plans and recovery of areas back to or from residential uses in over 75 per cent of the former and 35 per cent of the latter classification. Required construction unless controlled within reasonable and safe limitations will develop another era of white elephants, and blighted areas with added distress to tax-overburdened communities.

Not code revision but emergency regulation

The urgent program of speedy erection and minimum essential material consumption requires not a general revision of building codes but, as stated in the paper a complete emergency regulation. It should apply to essential defense construction only, be issued and directed by the proper national agency. The delegation of this work to the code committees of the various national technical organizations would result in delayed action. A competent committee of delegated representatives from each such organization should be appointed by federal authorities immediately to codify the basic technical specifications and their application.

Such proposals as the general use of reinforced concrete with war-time stresses, the general liberalizing of working stresses for all materials, the recently suggested increase of allowable unit stress for structural steel up to 28,000 psi for static loads combined with modern and to a large extent experimental methods of design will lead to structural and economical set-backs.

Experience indicates that decreases in specified load factors must be accompanied by corresponding decreases in the ignorance factor frequently prevalent in structural design. The writer is at present reenforcing a defense plant operating on a 24-hour schedule, seven days a week, in which production demands resulted in loading a reinforced concrete floor with 1,000 psf. designed for a load of 250 psf., an actual total load factor of 3 combined with occasional impact. The structure was saved from immediate collapse by the reinforcement of square twisted rods with a probable yield point of 50 to 55,000 psi., while designed with a working stress of 18,000 psi. Excessive deflection accompanied by cracking and spalling gave ample warning before collapse.

The use of high yield steels in reinforced concrete as demonstrated by the 1941 New York tests indicated safe working stresses of 24,000 lb., and if deflection and cracking are not deterrents even 30,000 psi. Such use must be tempered with judgement and controlled design. Monolithic structures frequently become permanent developments and should be treated accordingly. Consideration of the New York Code allowances for lower percentages of reinforcement in rodded columns, and thinner sections in structural steel design are also indicated. Increases in working stresses for structural steel design will follow consideration of the elastic stability requirements of the particular structure. The arbitrary recommendation of working stress with a factor of 1¼ on the yield point certainly demands careful scrutiny of the use of the structure. The steel frame of defense structures subjected primarily to wind loads might warrant such increase, but not the general case of "determinate static loads".

It is realized that engineers must do more, faster in the urgent industrial defense program. All materials must be considered. Timber in roof trusses, and on roof and floor decks can be erected on reinforced concrete as well as light structural steel frames with speed, economy, safety and flexibility. The timber can be treated for durability, as well as fire resistance. Heavy industrial structures can be economically and speedily erected at relatively low costs with untreated and treated timber, with full realization of their possible removal and salvage value in the more temporary structures. Reinforced concrete combined with timber lends itself economically to highway bridges and steel economy can be affected here as well as in the road construction itself.

The proposal for a national emergency code calls for immediate action in relation to all structural materials, individually as well as in combination. The desired objective would be expedited by the formation of a Joint Technical Committee operating under proper federal authority to correlate all the essential recommendations to the end that steel will be saved for its more critical war uses.

By CHESTER L. POST*

Every engineer will approve every reasonable effort to adjust design and construction for maximum utilization of critical materials, during the present emergency. Steel in any form is a critical material. Any revision in methods of design and construction, during the emergency, should be considered as a temporary expedient and should not permanently upset the orderly development of concrete design and construction.

The ACI and Joint Committee codes are the results of extensive research, study, and faithful effort of many capable and conscientious

*Chairman Advisory Committee on Engineering, Public Buildings Administration, Washington, D. C

men over a period of years. Any code must be a compromise of many viewpoints and must be applied to work which will be done by engineers and contractors ranging from those who are capable and conscientious to those who are incapable and careless. A code, therefore, should be sufficiently conservative to provide some margin of safety even under unfavorable design and construction conditions.

It will be unfortunate if the present emergency is used as a pretext to upset the ACI and Joint Committee codes. These codes do not fix loading, therefore, loads may be adjusted without requiring any material change in the application of the codes or any tables or diagrams based on the codes. The same is true with reference to the application of any recognized building code.

The following discussion is presented as an attempt to analyze conditions and to develop a method which will result in a reduction in the amount of reinforcing used in any particular project, without resulting in dangerous construction. In considering any change, it should be remembered that conditions will be less favorable than during normal times.

Unfavorable conditions

Design—Designs must be prepared hurriedly without the usual opportunity for careful study. Due to the great demand for engineers, some work will be designed and detailed by men of questionable fitness.

Inspection—Inspection may be done, in many cases, by inferior personnel and due to the emergency, even capable inspectors will be inclined to be less rigid in their requirements.

Construction—Requirements for speed may result in inferior materials and workmanship. Due to extreme demands for construction forces, the contractor's organization will not be of the same high quality as in normal times.

Emergency construction

Construction during the emergency should be based on the following factors, in order of their importance: (1) speed, (2) most productive use of critical materials, (3) economy.

1. *Speed*—For maximum speed, consideration should be given to simplicity of design. This will result in a saving of time and labor for building forms and the placing of reinforcement.

The use of readily available materials for forms, reinforcing, aggregates, etc., will permit greater speed.

2. *Most productive use of critical materials*—Metals, being critical, should be used as sparingly as consistent with safety.

3. *Economy*—Economy becomes a minor consideration as compared to the previously named factors.

Proposed method of design

Safety—Due to the unfavorable conditions for design and construction, during the emergency, any deviation from accepted practice should be carefully considered.

Safety factor is a term used to represent the ratio of ultimate loads to working loads, therefore, structures are designed to provide additional capacity to provide for unknown and indeterminate conditions.

This additional capacity may be provided by assuming additional loadings or by reducing design stresses. For the purpose of this discussion, the subject will be treated by increasing assumed loadings and later a factor will be applied to reduce the loadings to correspond with design stresses used in present practice.

Three factors are involved in providing an adequate margin of safety; (1) design, (2) construction, (3) overloads.

1. *Design*—Designs are largely based on imperical coefficients which may not provide uniform margins of safety for all conditions of loading. Errors may occur in calculations and detailing. *Assume 25 per cent increase* in loading to provide for safety in design.

2. *Construction*—The strength of the concrete may be lower than assumed. The reinforcement may not be placed accurately. The dimensions of concrete sections may be deficient. The curing of the concrete may be inadequate. *Assume 25 per cent increase* for construction.

3. *Overloads*—Structures or portions of structures are frequently subjected to live loads largely in excess of those assumed for design. *Assume 100 per cent increase* for overloads. The overload factor does not apply to dead loads, as the dead loads can be accurately determined.

Combining the above factors, the following results are obtained to represent ultimate conditions for design:

	Live Load	Dead Load
Design	.25	.25
Construction	.25	.25
Overloads	1.00	—
Loads assumed for design	1.00	1.00
Ultimate loads or safety factor	2.50	1.50

The above basis will result in a more uniform safety factor for all conditions of loading than current methods and deserve consideration for adoption in normal times. During the emergency, the additions for design and construction should not be reduced. A reduction may be made in the addition for overload. If the overload addition is reduced to 0.5, the following relationship will result:

	Live Load	Dead Load
Design	.25	.25
Construction	.25	.25
Overload	.50	—
Loads assumed for design	1.00	1.00
Ultimate loads or safety factor	2.00	1.50

To reduce loads so that usual working stresses may be applied, a factor must be applied to the assumed ultimate loads. A factor of .5 is recommended as representing a reasonable relation between yield point stress and working stress. The two cases then become for design loads.

1. .5 (1.5 DL + 2.5 LL) = .75 DL + 1.25 LL
2. .5 (1.5 DL + 2.0 LL) = .75 DL + 1.00 LL

The second formula is recommended for use during the emergency.

The attached table gives comparisons of current practice, formulas 1 and 2 based on: (1) resultant unit stresses for DL plus LL; (2) safety factor on basis of DL plus LL; (3) safety factor on basis of LL only.

The use of the formula .75 DL + 1.00 LL seems simpler to apply than changes in design stress, as no changes need be made in usual methods of design after adjusted loads are computed.

The following additional methods are recommended to save reinforcing.

(1) Increase normal values of d a minimum of 10 per cent.

(2) Use unreinforced footings and walls wherever feasible.

The ACI and Joint Committee codes provide methods for computing unreinforced footings. (Table G).

(3) Where special structures are designed for loads such as tanks or machines where position and maximum loads do not vary, the loads may be treated as dead loads.

(4) Use types of construction which require minimum reinforcing.

(5) Use high strength concrete in members for which amounts of steel are governed by compressive, shearing and bond stresses.

(6) Increase the sizes of columns as much as conditions will permit.

(7) Use prestressed reinforced concrete for circular tanks and circular roof structures. Considerable saving in steel will result and cracking will be minimized.

The above methods will result in more balanced designs insofar as safety is concerned and will result in the minimum amount of changes in office practice.

Acknowledgment is made for helpful suggestions made by other members of the society.

TABLE G—COMPARISONS

Unit Stresses Based on Intermediate Grade of Steel
For other grades of steel the changes will be proportional

DL / LL	.25 / 1	.5 / 1	1 / 1	2 / 1	3 / 1	4 / 1	5 / 1
Current Practice	20,000	20,000	20,000	20,000	20,000	20,000	20,000
1) .75 DL + 1.25 LL	17,400	18,500	20,000	21,820	22,860	23,500	24,000
2) .75 DL + 1.00 LL	21,050	21,820	22,860	24,000	24,620	25,000	25,260
Safety Factor Based on DL + LL							
Current Practice	2.0	2.0	2.0	2.0	2.0	2.0	2.0
1) .75 DL + 1.25 LL	2.3	2.16	2.0	1.83	1.75	1.70	1.67
2) .75 DL + 1.00 LL	1.9	1.83	1.75	1.67	1.63	1.60	1.58
Safety Factor Based on LL only							
Current Practice	2.25	2.5	3.0	4.0	5.0	6.0	7.0
1) .75 DL + 1.25 LL	3.06	3.01	3.0	3.04	3.12	3.22	3.33
2) .75 DL + 1.00 LL	2.01	2.03	2.06	2.17	2.28	2.40	2.52

The above values are based on relative loads and do not take account of the negligible changes which would be caused by variations in *jd* due to changes in per cent of steel

1) Is equivalent to using 26,666 psi. for DL, SF = 1.5
and 16,000 psi. for LL, SF = 2.5

2) Is equivalent to using 26,666 psi. for DL, SF = 1.5
and 20,000 psi. for LL, SF = 2.0

By W. C. SPIKER*

The writer is strongly in favor of designing both reinforced concrete and structural steel on a basis of dead load plus at least one and one half times the live load at a stress appreciably under the elastic limit of the material.

At a convention of The American Institute of Steel Construction about twenty years ago the writer proposed the above method and at that time pointed out that a constant unit stress for girders and beams carrying only dead load as compared with girders and beams carrying a high percentage of live load is inconsistent.

Recently, through an associate, the writer was glad to hear that by reason of the War there is now some prospect of discontinuing the waste which has been going on through the decades.

For the purpose of simplifying the design work and avoiding confusion the following formula or some similar formula is proposed:

$$\text{Design Load} = .7\ (\text{D.L.} + 1.6\text{x L.L.}).$$

*Engineer Adviser to Public Buildings Administration, Washington, D. C.

Possibly a simpler formula for gradually decreasing the actual unit stresses under D.L. only to the present existing unit stresses under D.L. + say 350 to 450 per cent L.L. can be devised which will also permit calculations to be carried on as at present and not change present working unit stresses.

By HALE SUTHERLAND* AND BRUCE JOHNSTON†

Mr. Bertin has written a clear and concise article on the important question of conserving steel in the emergency. The writers have no arguments against increasing stresses in the use of high yield-point steel provided that it is understood and expected that in some cases such stresses will produce visible cracks. Such cracks will be the result of stress-producing loads in addition to any shrinkage or temperature cracks which may also develop. Cracks in concrete will in some cases be of little importance, particularly in some structures of a temporary or emergency nature.

There is likely to be danger of carrying over into post-war practice design procedures which have been adopted specifically for the emergency. This may or may not be desirable, and emergency design practices must be carefully re-evaluated before they are allowed to apply to permanent structures.

This word of warning is of importance, particularly since Mr. Bertin believes present standards to be unduly conservative. Although this may be true in some respects the writers are unable to agree that this estimate of present day codes is generally the case.

By CARLTON S. PROCTOR‡

Mr. Bertin's paper could not be more timely. Engineers charged with the responsibility for structural design must awaken to the fact that the national emergency requires all possible conservation of steel, and that a waste of steel today is distinctly a national dis-service.

There can be no question but that steel consumption can and must be greatly reduced, and that this can be largely accomplished through intelligent design and code modifications which will take cognizance of the fact that many codes are unnecessarily extravagant in their requirements and that emergency codes should immediately be adopted permitting high working stresses for temporary structures and encouraging substitutions wherever feasible.

The engineer's responsibility is to include in his design, substitutes for steel wherever permissible and to assure that the usual over-design,

*Head of the Department of Civil Engineering and Director of Fritz Engineering Laboratory, Lehigh University, Bethlehem, Penn.
†Associate Director of Fritz Engineering Laboratory, Lehigh University, Bethlehem, Penn.
‡Moran, Proctor, Freeman & Mueser, Engineers, New York, N. Y.

or factor of safety (frequently a factor of ignorance) be reduced to a safe minimum.

The Steel Conservation and Reclamation Committee of the Engineers Defense Board has recently made recommendations to the OPM, which if adopted, will result in conservation of steel and use of substitute materials. The emergency code included in Mr. Bertin's paper is a part of that report. It is expected that future reports of this committee will include additional constructive recommendations, and it is to be hoped that all engineering bodies whose membership includes designers of structures and framers of codes, will adopt revised standards of design and of code requirements in an effort to help to meet the present national shortage of ferous metals. The publication of the Steel Conservation and Reclamation Committee's report* has already brought forth a great many favorable comments from governmental and private designers. Quoting from one letter received on the subject, the writer states: "I can positively state that I have found in most defense drawings a waste of reinforcing steel amounting in most cases to not less than 30 per cent."

This and other letters and comments point out numerous procedures for the conservation of steel in structural design which were not covered in the first report of the committee, and indicate the large opportunities for steel conservation or substitution which are open to the designing engineer.

It is very much to be hoped that through publications such as that of Mr. Bertin's paper, and through code revisions and committee activities of societies such as the American Concrete Institute, the engineering profession in this country will make its full contribution to the national emergency.

By LANE KNIGHT†

The foregoing discussion has undoubtedly been interesting but most of the exceptions to Mr. Bertin's paper were based on possible complications in multiple story construction. Actually, however, the bulk of our present construction and probably all of our future defense projects will be relatively simple one story structures, so the criticisms are hardly pertinent to today's problem.

We are all familiar with the typical defense plant which is little more than a floor and roof without any particular complex design problem save perhaps the problem of spans. Now the Institute Committees have been criticised in the past for the time required to bring forth a definite recommendation in reports, but as time is now the essence of our war effort,

*Civil Engineering, February 1942.
†Vice-President Master Builders Co. Ltd., Toronto.

I believe we would be doing a worthwhile service if the various Committees could immediately cooperate in the development of a simple basic specification for a typical defense plant. The specification of course would be based on reducing all reinforcing steel to an absolute minimum but compensating for this reduction by an increase in the compressive strength of the concrete. This is in line with the interesting figures shown by Mr. Germundsson indicating savings effected by increasing the compressive strength of the concrete.

I sincerely feel that such a specification would be of tremendous value as a guide to the engineers and architects who are engaged in defense construction. Possibly a definite job specification might not be practical for various reasons but it would seem that the Institute could and should by an elaboration of Mr. Bertin's theme suggest in a published report possible ways and means of conserving steel in defense construction.

CLYDE T. MORRIS*

This subject was discussed at the February meeting of Committee XV (Iron and Steel Structures) of the American Railway Engineering Association of which I am a member, and I was recently called to New York by the American Standards Association as a member of a committee to frame specifications for the WPB for the use of higher working unit stresses in structural steel for buildings, as a means of conserving steel in the present emergency.

Highway departments and the railways are greatly hampered by the necessary priorities required for steel for buildings, bridges and reinforcement. As stated by Mr. Bertin, the use of reinforced concrete, properly designed, in place of structural steel will free considerable steel for use in the war effort; but I believe that we may go much further in that direction by eliminating the reinforcing altogether in certain types of structures.

The Government itself, in war projects which have A-1-A priorities, is in many cases using steel with a prodigal hand. Engineers have been schooled in the use of reinforced concrete until the design for almost any structure which may not be termed "mass concrete" will show networks of reinforcing steel where by no stretch of the imagination can any tensile stress be found for it. This conservatism is probably justified in normal times and in structures planned for unknown future loadings. But in this emergency there is a large field for plain concrete which, in the rush for quick construction, seems to have been forgotten.

(a) The "igloo's" used in the numerous munitions plants for storage of explosives are reinforced with networks of steel in each face, both of the arch-barrel and the end-walls. If the shape of the arch barrel is

*Dept of Civil Engineering, Ohio State University, Columbus, Ohio.

properly formed so that the thrust lines for the worst conditions of loading remain within the middle third of the cross section, no reinforcing steel is necessary. To accomplish this it may be necessary to increase slightly the amount of the concrete used. In the head walls, no one would question the stability of 13 in. brick walls for a height of 15 ft. Why not a 12 in. plain concrete wall?

(b) There are hundreds of stone masonry arches which have been doing service for centuries. There are also many plain concrete arch bridges constructed from 10 to 40 years ago which have shown no signs of distress from loads or temperature changes. A notable example is the Rocky River Bridge at Cleveland, Ohio, built in 1910 with a clear span of 280 ft. Many others might be enumerated.

(c) Gravity type foundations, dams and retaining walls are perhaps more effective in their functions than those of reinforced concrete. Why not extend their use in this emergency?

I have given considerable study to the use of plain concrete for structures such as those cited above and find that, with proper design, igloos and arch bridges can be built, practically without steel, by increasing somewhat the amount of concrete used. In the case of arch bridges with ratio of rise to span of one-half, the increase in concrete section will be slight. As the ratio of rise to span decreases thicker sections will be required.

The elimination of the steel, especially in thin sections, will be conducive to better concrete, as the placing and spading is much easier and the amount of mixing water may be reduced. The total cost may be increased or not, depending upon the relative cost of steel and concrete in each particular case. But in the present emergency cost is not a determining factor. Conservation of steel for guns and ships and tanks is paramount.

Such further discussion of Mr. Bertin's paper as is received (in triplicate) by the ACI Secretary by June 1, is scheduled for publication in the JOURNAL Supplement concluding Proceedings V. 38

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Concluding Discussion of a paper by R. L. Bertin:

Saving Steel in Reinforced Concrete Design*

DISCUSSION BY EDWIN F. ALLBRIGHT and AUTHOR

By EDWIN F. ALLBRIGHT†

Interesting as is the subject discussed by Mr. Bertin, I took exception to his statement that "little can be said regarding the relative steel consuming characteristics of the various types of floor and roof construction."

(In other discussion Mr. Germundsson tabulates‡ steel quantities in lb. per sq. ft. of floor area for 7 types of floors, taken from design studies for 20-ft. span and a 100-lb. loading. The table clearly points out the superiority of the flat slab design, which requires only 2.13 lb. of steel per sq. ft., which is considerably less steel than the next lowest type, and is also the most economical floor design system.)

The Grid system of reinforced concrete construction has often been in competition with the solid flat slab construction. The savings in general have run from 20 to 30 per cent in concrete and from 10 to 20 per cent in reinforcing steel. Under present conditions, where the saving of reinforcing steel is of paramount importance, we recently have been making our comparisons where the amount of concrete in the Grid system is the same as in the solid flat slab, thereby making the entire saving in the reinforcing steel.

Some calculations made on a solid flat slab for an interior panel 20 x 20 ft. and for a live load of 100 lb. per sq. ft. gave us for the weight of steel 2.13 lb. per sq. ft.† This design was based on a solid slab 7½ in. thick and a drop panel 7 by 7 by 3½ in. and a column cap of 4 ft.-6-in. diameter.

Calculations were also made on the Grid flat slab, having the same amount of concrete, for an interior panel the same size and live load,

*ACI JOURNAL, Feb. 1942; *Proc.* V. 38, p. 281: discussion, p. 288-1 to 36, Apr. 1942 Jl. The announced schedule called for closure on the discussion of Mr. Bertin's paper June 1 and publication in this *Supplement*. The timeliness of the subject and the many contributions to the discussion at the 38th Annual Convention, Chicago, Feb. 20, prompted the publication in the April JOURNAL of discussion then available.

†Secretary and Engineer, Grid Flat Slab Corp., Boston, Mass.

‡ACI JOURNAL, Apr. 1942, p. 288 - 12.

basing the design on 8-in. domes plus 3-in. of concrete topping, a drop panel 7 x 7 ft. x 4 in. and a column cap 4 ft. 6 in. in diameter. The weight of reinforcing steel in the Grid slab figured to 1.41 lb. per sq. ft., showing that the solid flat slab requires 50 per cent more reinforcing steel.

The saving is effected without increasing the allowable stresses or decreasing the live or dead loads; in other words, without making any changes in existing codes.

AUTHOR'S CLOSURE

The writer is indebted to those who have been kind enough to discuss the paper, thereby adding so much to its value.

Since the writing of the paper much has happened to crystallize the concept of steel saving into actuality. The culmination is found in the issuance by the War Production Board of a set of "National Emergency Specifications for Design of Reinforced Concrete Buildings," the use of which is prescribed "for the War Production Board and for the Army, Navy, Maritime Commission, Reconstruction Finance Corp., National Housing Agency and for all other departments and agencies in respect to war construction and the financing of war construction.*"

*See ACI JOURNAL, Nov. 1942—EDITOR.

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Architectural Concrete on the New Naval Medical Center*

BY HUGO C. FISCHER†
Member American Concrete Institute

SYNOPSIS

This paper describes certain features of the design, production, and setting of the precast reinforced-concrete, exposed-aggregate panels used for the exterior facing of the new Naval Medical Center at Bethesda, Md.

INTRODUCTION

The outstanding feature of the new Naval Medical Center at Bethesda, Md. is the use of an architectural concrete facing for practically all of the buildings. This facing consists of precast reinforced-concrete exposed-aggregate panels set for the most part in a manner similar to natural stone and backed with brick work. (Some panels were also applied on concrete basement walls in place, but this procedure, as will be mentioned later, is not recommended.) These panels were used on the eight principal buildings of the medical center and will be used on additional buildings soon to be placed under contract. The buildings now completed on which the panels are used include: Main building (Fig. 1 and 3) with a 20-story axial tower, a 4-story north wing for the Dental School, a 4-story south wing for the Medical School, and a 3-story east wing containing operating rooms and accessory facilities; Subsistence and Recreation building (Fig. 2) adjoining the east wing of the Main building, which provides galley, messing, storage, and recreational facilities including a large auditorium.

Nurses' Quarters (Fig. 4), northeast of the Main Building group, providing living facilities for 70 nurses; Hospital Corpsmen's Quarters (Fig. 5), south of the Main Group, furnishing living and recreational facilities for 280 corpsmen; two ward buildings (Fig. 6), directly south

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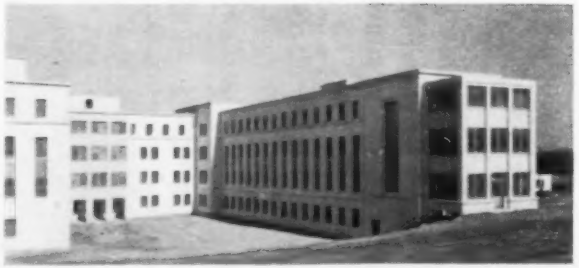
†Captain, CEC, U. S. Navy; Bureau of Yards and Docks, Washington, D. C.; in charge of construction new Naval Medical Center.

Fig. 1 (above)—Main building group, looking northeast, of Naval Medical Center, Bethesda, Md.

Fig. 2 (below)—The Subsistence and Recreation Building adjoins the east wing—laundry and garage are in left foreground

Fig. 3 (right)—West front of Main Building and the Tower



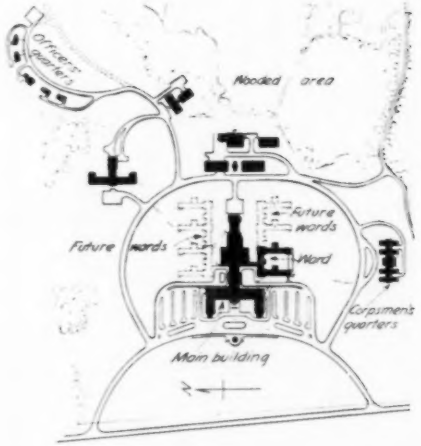


Fig. 4 (top left)—Nurses' Quarters (rear view) with living facilities for 70

Fig. 5 (center)—Corpsmen's Quarters (front view) with living facilities for 280

Fig. 6 (bottom left)—Two of the Ward Buildings—south of Main Building

Fig. 7 (above)—Ground plan of the Naval Medical Center—Scale (approx.) 1200 ft. to in.

of the operating wing of the Main building and the Subsistence and Recreation building and connected thereto by corridors (two additional ward buildings on the south and four on the north side are contemplated); laundry and garage and central heating and refrigeration plant, located to the east of the Main building group (Left foreground of Fig. 2). The entire medical center is shown in a ground plan in Fig. 7.

Work was started by the Bureau of Yards and Docks on the functional layout of the new establishment, in collaboration with the Bureau of Medicine and Surgery of the Navy Department in 1938, and by the spring of 1939 had advanced to the stage of structural and architectural design. Paul P. Cret, Philadelphia, was retained as consulting architect, and later when it was found that due to the great complexity of the project, the Bureau of Yards and Docks design force was unable to complete all buildings in time, Mr. Cret's office was also commissioned to prepare the contract drawings for the ward buildings, nurses quarters, and corpsmen's quarters.

Brick or limestone was first contemplated for these buildings, but it appeared on further consideration, that the architectural effect desired could be best achieved by the use of exposed-aggregate concrete panels, similar to those used by the Navy on the new ship model testing plant at Carderock, previously described in this JOURNAL.* However, on these taller buildings, it was decided to set the panels with a brick backing instead of using them as face forms for concrete structural walls as was done at Carderock. This change was made principally to facilitate progress of construction, since using the panels as face forms involved delay to install back forms and to pour the backing concrete after setting each course of panels before the next increment in height of the building could be undertaken, whereas the brick backing could follow directly around with the setting of each panel course.

A STUDY OF PRECAST CONCRETE

The principal question raised in connection with the use of the exposed-aggregate panels was that of durability. The Chief of the Bureau of Yards and Docks, Admiral Moreell, directed the writer to make an investigation and submit a report.

Visits were made to numerous buildings with architectural concrete exteriors, tests were arranged with the Bureau of Standards, and information obtained from the Portland Cement Association, F. R. McMillan, Director of Research, and C. G. Walker, Secretary of the Cast Stone Institute, being particularly helpful. John J. Earley, Past President of

*"The Navy's New Ship Model Testing Plant," Hugo C. Fischer; ACI JOURNAL, Apr. 1939; *Proceedings* V. 35, p. 317.

the Institute and originator of this type of architectural concrete facing, also contributed extensively of his time and furnished invaluable comment and information on the design of the panels. The report, in three parts, is abstracted as follows:

Permanent good appearance

Precast exposed-aggregate panels are nothing more nor less than a very high-class concrete with a special face treatment to expose aggregates selected for appearance. As such it is subject to comparison with other architectural concrete, much of which has been unsatisfactory. Many architects who employed cast stone some years ago have gone back to brick and natural stone because of cracks, discolorations, and spalls which occurred in the cast stone of that period. The best such concrete has endured in spite of severe exposure. These examples are proof that concrete can be made to withstand almost any exposure to which an architectural building material might be subjected.

Underlying causes for disintegration of concrete exposed to weather are stresses due to volume changes through variations in temperature and moisture and the freezing of water in the pores of pervious concrete. The precast panels for the new Naval Medical Center are especially designed to meet these stresses as follows:

(1) They are cast in thin diaphragms (2½ in. thick) of limited size (few more than 4¼ x 9 ft.) with free edges. Stresses due to volume changes are practically two-dimensional instead of three-dimensional, as in heavy monolithic construction.

(2) They are heavily reinforced with a welded mesh of quarter inch diameter steel wires on 4-in. centers each way, this reinforcement being made of galvanized wire and dipped in hot pitch after all forming, cutting, and welding is completed. Each welded intersection serves as a nucleus around which the mass movement of the concrete is restricted and thus prevents an accumulation of stress tending to produce cracks in large slabs.

(3) The mix of the concrete (94 lb. cement, 105 lb. fine aggregate, 305 lb. coarse aggregate) produces a dense, strong, and low water-absorptive product. This concrete averages about 6,500 psi compressive strength at 28 days. Tests made by the Bureau of Standards on specimens of panels used at the Model Basin, Carderock, indicated an average water absorption of only 4.7 per cent in five hours boiling. Stresses due to shrinkage of concrete in setting are reduced by using a low water-cement ratio (5 gal. per bag of cement).

(4) Cracking of concrete in monolithic buildings is often due to restraint caused by the foundations, the floor and roof systems, the interior

walls, and increased stiffness at the corners of walls where the return adds to the rigidity. The method of mounting the precast panels as an exterior facing, substantially free of the structural elements of the building, allows deformations in the frame of the building as a whole to take place without bringing undue stresses on the panels, since the movement can be taken up in the plastic caulked joints between the relatively small sections of facing.

(5) These panels are manufactured in plants under ideal conditions for close control of the product. The specifications set forth the requirements in great detail, and qualified government inspectors watch and check every operation constantly to insure careful compliance.

Other elements considered on the wearing qualities of precast panels were:

(*a*) *Color permanence.* The aggregates exposed on the face of these panels are natural quartz, serpentine, and marbles which retain their color for ages.

(*b*) *Discoloration.* Any light-colored material will discolor, especially in industrial centers where the air is laden with soot, the degree of dirt adherence depending on the smoothness and density of the surface. It is believed that the exposed-aggregate panels will discolor less and will clean more easily than the more porous natural stones, but will discolor somewhat more under the same conditions than a polished marble or granite.

(*c*) *Crazing.* This was a common defect on cast stone trim which was made with a very fine aggregate and float rubbed to imitate natural stone. Such rubbing produced a skim coat of mortar which generally crazed and sometimes flaked and produced a most unsightly condition. On the exposed-aggregate panels the surface mortar is first removed by a sanding machine or wire brushes and then further etched off with acid.

Tests by the Bureau of Standards

Freezing and thawing tests were made of four prisms (2x2x12 in.) and six cores (2-in. diameter and 2 in. high) cut from a section of the precast exposed-aggregate panels used at the Naval Model Basin, Carderock. The test specimens were first immersed in water at 70° F. for 48 hours and then passed through cycles consisting of 18 hours of freezing at 18 to 24 F. (all specimens being frozen in one-half inch of water) and 6 hours of thawing in water at approximately 70 F.

The ten test specimens passed through an average of 307 cycles of freezing and thawing before they reached a condition such that the value of the panel for facing would be utterly destroyed. The Bureau rougly estimated that four cycles of such freezing and thawing tests

were about equivalent to an average year's weathering in this climate, although it may be considerably more or less because the conditions of the tests in which the specimens were fully saturated are different from those occurring on a building where vertical surfaces might rarely be exposed to any considerable degree of saturation when freezing occurs. On the basis of four cycles being equivalent to a year of weathering, the effective life on a building would be about 75 years. While laboratory freezing and thawing tests were considered to give the best indications now available on the durability of concrete, it is recognized that they are far from reproducing the conditions in nature.

The Bureau of Yards and Docks then arranged with Mr. Earley to prepare test specimens representing 17 different mixtures from specifications of the Navy Department, which were then given accelerated freezing and thawing tests by the Bureau of Standards.

The specimens consisted of precast slabs 2½ x 12 x 20 in. and molded bars 2 x 2 x 10 in. Specimens 2 x 2½ x 12 in. were cut from the slabs for the freezing and thawing tests; the molded bars were tested as received; 2-in. cores were cut from the slabs for absorption and compression tests. The weights per cubic foot were determined from the weights of the slabs in air and submerged in water after removal of the test specimens.

One molded bar and one bar cut from a slab of each mix were subjected to cycles of freezing at −10 F., starting at the age of 35 days, and thawing at 70 F. The modulii of elasticity of the specimens were determined by the sonic method prior to the freezing and thawing treatment and again after every fifth cycle and the change in modulus reported as the percentage decrease from the initial value. The types of mix and results are abstracted as follows:

Mix 1. 94 lb. cement, 90 lb. fine aggreate, and 300 lb. coarse aggregate was in the same proportions as used on the Naval Model Basin.

Mix 2. Same weights of aggregates, cement reduced to 80 lb.

Mix 3. Same weight of cement and coarse aggregate as Mix 1, increasing the fine aggregate to 115 lb.

Mix 4. Same as Mix 1, but further reducing cement to 70 lb.

Mixes 5, 6, 7, and 8 were the same as 1, 2, 3, and 4 respectively but with stearate added.

The test results indicated that a reduction in cement content also reduced the freezing and thawing resistance, but that an increase in fines improved the mix used at Carderock, as Mix 3 showed the greatest resistance. The data obtained indicated that the addition of stearate was not particularly effective.

Mixes 9, 10, and 11 were then made up with the higher proportion of fine aggregate, viz. 115 lb., holding the coarse aggregate at 300 lb. and varying the cement content, using 70, 80 and 105 lb. respectively. Again the mix with the highest cement content had the best freezing and thawing resistance, and the two lower cement content mixes showed definitely unsatisfactory resistance.

Mix 12 was then made up as a control mix with 94 lb. of cement, 110 lb. of fine aggregate, and 300 lb. of coarse aggregate. *Mixes 13, 14, 15, 16, and 17* were the same as No. 12, with the addition of proprietary integral waterproofing admixtures.

Mixes 15 and 16 weighed only 127 lb. per cu. ft. as compared with 140 lb. for the standard control Mix, 12. (These mixes were all made with quartz aggregates throughout, instead of quartz aggregate facing and sand and gravel aggregate backing, as in the actual panels, which apparently accounts for the low unit weight of the control sample.)

Mix 16 had the strange characteristic of actually showing a continuing increase in the modulus of elasticity up to 95 cycles of freezing and thawing when the tests were stopped. However, this mix tested only slightly more than half the compressive strength of the control sample at 28 days.

Mix 15 also showed a decrease of only 6 per cent in modulus of elasticity at 95 cycles, whereas the control specimen showed a decrease of 50 per cent at the end of 30 cycles, and this characteristic of reduction in modulus of elasticity by 50 per cent at the end of 10 to 30 cycles was also found with all other mixes. However, the compressive strength of Mix 15 at 28 days was only three-quarters that of the control sample.

With the exception of Mixes *15 and 16*, which started with a low strength, the use of the integral waterproofing admixtures showed no improvement in strength, absorption, freezing and thawing resistance or any other element indicating improved durability, and consequently no waterproofing admixture was used on the panels for the Medical Center. It is recognized that the mixture for these panels is a special one, and tests on other concrete mixtures and aggregates might produce more favorable results with the admixtures tested.

Inspection of buildings with exposed aggregate panels or other architectural concrete

Inspection was made of nine structures in the Washington area on which exposed-aggregate concrete, as produced by the Earley studio, was used, ranging in age from the Meridian Hill Park retaining wall built in 1915 to the Normandy Building built in 1938. While some minor defects were noted, the work as a whole was in remarkably good condition, and all precast work was in excellent condition.

The exposed-aggregate work at the Naval Model Basin, Carderock, which had then passed through its second winter, was given a particularly detailed inspection, and while a considerable number of minor defects were listed, the appearance of the buildings as a whole was outstandingly good. Most of the defects noted were not observable from a distance of 50 feet, and the cracks were mainly so small as to be self-healing and in no case affecting adversely either the structural strength or functions for which the buildings were constructed. These cracks were apparently due to three causes:

(1) Excessive strains in bringing the edges of the large, and in some cases complex, sections of panels to the exact line required to meet the faces of adjoining panels and, in the earlier stages of the work, a few cases in which excessive pressures were developed by pouring the vibrated backing concrete too high against these panels as a form.

(2) The volume changes and cracking of the monolithic structural backing concrete to which the panels were firmly bonded. In this connection it is noteworthy that where joints in successive courses of panels were offset as much as 9 in. and the backing concrete developed a continuing crack opposite one joint, the strength of the panels was sufficient to carry the movement the distance of the offset to the next panel joint instead of cracking through the panel directly opposite the crack in the backing concrete.

(3) Strains and warpage due to handling. The panels were manufactured in New Haven, Connecticut, shipped by freight to Bethesda, Maryland, and trucked from thence to the site. It is quite possible that some of the panels, particularly those of necked-in irregular sections or with deep right-angle returns were strained to incipient cracking by this extensive handling.

A number of buildings and structures were also inspected in the Chicago area, including the Bahai Temple constructed by Mr. Earley from 1932 on. This beautiful building represents the outstanding achievement to date in the use of concrete as an architectural material. The earlier part of the work had then passed through six winters without showing any sign of deterioration. Some excellent examples of cast stone work were noted at the Soldiers Field Stadium, Portland Cement Association Building, and the Bankers Building built in the mid-twenties. At the P. C. A. Building slight erosion was noted on window sills and water tables and several small rust spots where moisture had reached reinforcement found to have been placed accidentally within less than one-half inch of the face of the stone. Reinforcement for the Medical Center panels is zinc-coated, pitch-dipped, and kept back accurately to a minimum of one inch from the face.

Fig. 8—(a) Casting, finishing and curing exposed aggregate panels (b) final inspection in storage yard

PRODUCTION OF PANELS

Work was started on the construction of the buildings at the Medical Center in May, 1940, under general contract with John McShain, Inc., Philadelphia. The production of the exposed-aggregate panels was undertaken by the Federal Seaboard Terra Cotta Corp. at one of its plants in Woodbridge, New Jersey, which had formerly been used in the production of architectural terra cotta. This company had no previous experience in producing concrete products, but had extensive design facilities and a management with wide experience in architectural construction. The Bureau of Yards and Docks assigned to the plant a competent senior inspector, qualified by experience on the production of the panels for Carderock, and two assistant inspectors, to watch and check every operation for careful compliance with the requirements. The senior inspector rendered material help on the layout of the plant and development of processes, and, last but not least, in the development of the necessary craftsman's skill in the workers, especially for the finishers.

The plant and its operation

The plant included a mold shop equipped with standard power woodworking equipment, a reinforcing steel fabricating shop including a wide bending brake and Taylor Winfield portable electric welder, a dip tank and finished cage storage room; a 2-bag mixer plant for the face concrete and a 1 cu. yd. per batch plant for the backing concrete, both completely equipped for accurate weight proportioning of all elements.

The casting, finishing, and curing departments were arranged as a straight continuation of the mixing plant and served by three lines of overhead cranes, extending through the end of the building to the outside storage yard (Fig. 8a and b). An outside overhead bridge crane, operating transversely to the three building cranes picked up and loaded the panels from the storage yard to cars.

The schedule required delivery of approximately 400,000 sq. ft. of facing in ten months or at the rate of 10,000 sq. ft. for a five-day week. The plant was laid out on an anticipated production of 2,000 sq. ft. per day, which required that each casting space be used daily. This meant that the casts had to be strong enough for removal from the forms in 16 hours, leaving 8 hours of the 24 for reconstruction of the molds, placing reinforcing cages, and pouring the casts to be pulled the following day.

Curing temperature

To keep up this rate of progress it was found necessary during the winter months to maintain a temperature of not less than 65 F. in the

casting and curing section throughout the day and night, and this minimum temperature was assured by placing recording thermometers in various parts of the plant. The hours required between pouring and lifting were found to vary with temperature as follows: 50 F., 20 to 24, 60 F. 16 to 18, 70 F. 14 to 16, 80 F. 12 to 14.

Aggregates

The aggregates used for the facing of all but a relatively small proportion of colored panels consisted of a mixture of 80 per cent opaque and 20 per cent translucent quartz. These were shipped in 100-lb. bags from Maryland quarries to the plant in New Jersey, ready crushed and sieved to the following gradations:

Per Cent Retained on Each Sieve

	Fine Aggregate	Coarse Aggregate
$\frac{5}{16}$-in.		0
No. 4		57.50
No. 8		95.0
No. 16		100
No. 30	1.25	
No. 50	70.00	
No. 100	91.25	
Thru No. 100	8.75	

To produce this gradation required special equipment and recirculation through an impactor, breaking the quartz as little as possible on each passage to avoid an excessive overproduction of fines to be wasted. The fines produced by the original crushing equipment at the quarry had an excess of iron, which turned the face to a rusty color, and it was necessary to use a magnetic separator until the improved equipment was installed.

The gradation of the backing aggregate averaged slightly coarser; a standard, well-graded, washed concrete sand being used for fine aggregate and a washed gravel for the coarse aggregate, all passing through a $\frac{1}{2}$-in. sieve and retained on a No. 16 sieve.

Reinforcing treatment and fabrication

As previously stated, all mesh reinforcement was dipped in hot pitch after all cutting, forming, and welding of the cages had been completed. Experiments were made with various asphalts, asphalt cutbacks, asphalt and tar varnishes, coal tar pitches, etc. and it was found that a so-called "steep roof" coal tar pitch with a melting point of 160 F. was the most satisfactory. The best temperature for this dip was found to be 320 F. with the mesh left in the dipping tank from two to three minutes to

heat through in order to get a minimum of globules at intersections. This coating would harden within two or three minutes after the cage was hoisted out of the dip. The dipping tank was 8 ft. wide and 12 ft. long and heated with a gun type oil burner with automatic temperature control. 85 bbl. or 27,000 lb. of steep roof coal tar pitch were used to coat the reinforcement for the 400,000 sq. ft. of panels.

Molds and casts

About 500 different shapes of panels are included in the facing for the different buildings. Considerable ingenuity was required in the design of these molds and high-class joiner work with heavy rigid construction was necessary to maintain the close dimensional tolerances allowed. Some of the more complex molds were made in plaster.

Skill was also required in fabrication of the wire mesh cages to fit the intricate design of some of the molds. Shop order cards were used which showed the details of the form on one side and of the corresponding reinforcement cage on the other.

Wooden forms were at first lined with Masonite. However, this material could not be obtained in width greater than four feet, and there was difficulty in preventing leakage which showed a line on the casting where it was necessary to butt the masonite liner, since most of the panels are more than four feet in width. Later, a similar material was used which could be obtained in widths up to 8 ft., but the latter material gave off more color than masonite, and it was found necessary to shellac this material before each cast instead of only on first use as was the case with Masonite. In any case it was necessary to grease the mold before each cast to prevent sticking. A mixture of a pint of motor oil and 6½ lb. of ground soapstone added to 10 qts. of kerosene was found satisfactory.

Plaster molds were treated with an application of soap solution (2 lb. of soap dissolved in a gallon of boiling water), and an application of aluminum sulfate solution next (11 oz. aluminum sulphate dissolved in a gallon of water at 70 F.) to nullify the suction effect of the plaster; followed by greasing as for the wooden molds.

Finishing casts

As soon as the panels could be safely picked up from the molds, they were placed on easels, and the face aggregate was exposed by electric-driven disk sanders (Fig. 9). They were then cleaned with a muriatic acid wash, hosed off, and covered with double thicknesses of muslin, which was kept wet constantly for five days. Final inspection was made in direct daylight and such retouching with light acid done as might be required before shipment. Minor damage in transit was repaired by expert workmen at the site.

Fig. 9—Facings exposed by electric driven disc sanders

In this connection it should be noted that there is a general tendency among workmen to use strong acid, up to 1 part of acid to 3 parts of water, to expose the aggregate by chemical action and save work on exposure by mechanical means with the sanding machines. This tendency has to be looked out for, as even if the strong acid is promptly and thoroughly hosed off with clean water, there is some deterioration of the surface, and the texture is no as good as when all the surface mortar has been properly removed by mechanical means and the acid wash with a strength of not more than 1 to 5 is used only to clean up the surfaces of the mechanically exposed aggregate. Here as in other cases there is no satisfactory substitute for "elbow grease".

While the specification provided that not more than five gallons of water should be used per bag of cement, it was found necessary to use about six gallons for the facing mix to insure a satisfactory surface. But the quartz mixture bled fast, and by vibration and working with trowels about a half gallon was removed before the drier backing mix was placed.

Colored panels

Colored panels were used for spandrels of the main building group and for trim of entrances at the Nurses' and Corpsmen's Quarters. These colors were obtained entirely by the use of colored aggregates with the exception of the dark spandrels on the Tower of the Main building. The aggregates used for the Tower spandrels were the same

Virginia serpentine stone as used on the spandrels of the wings. However, the fracture surfaces of the crushed stone were, as is general with any architectural building stone, of a lighter cast than the polished surface of the stone as used in the wings, which was almost black with a slight dark green cast. Accordingly, it was found necessary to add a small percentage of black iron oxide to the mix for the Tower spandrels in order to obtain the desired degree of dark color and still retain the dark greenish cast of the serpentine stone. Standard terrazzo aggregates were used for all other colored panels without the addition of any pigment. This, of course, avoids any possibility of pigments fading or, if used to any considerable extent, affecting the quality of concrete.

DESIGN OF PANELS

The walls of the buildings are of three different types: (1) supported on the structural steel framing of the Tower; (2) free standing but anchored to the concrete structural frame for the wings of the Main building and all other buildings except quarters; (3) wall bearing construction for the nurses and corpsmen's quarters. The design and mounting of the panels also varied, being of three types as described hereinafter.

Fig. 10 shows the method of mounting the precast panels on the spandrel beams of the steel structural framing of the Tower as originally proposed. In actual construction it was found simpler to run the reinforcement or setting lugs directly through the 9-in. gaps indicated on the drawing to be filled with brickwork, to drill holes for the anchorage dowels in the spandrels within the area of these openings, and then fill the gaps in the lugs with concrete instead of brick at the time the grout filler between the facing of spandrels and the back of the panels was poured. This provided a very effective anchorage and avoided the difficulty of matching holes for the dowels in the structural concrete and panel lugs. Fig. 11 and 12 show the method actually used on the Tower.

Fig. 13 shows a typical setting of the precast panels for the walls of the wings of the main building, ward buildings, and other three-story construction. These walls are free standing and not mounted on the structural concrete frame.

Fig. 14 shows the method of securing precast panels to the concrete walls of basements previously poured in place, as originally designed. Various modifications were used in actual construction, including the provision of slotted inserts, but none were entirely satisfactory. The insert rods were difficult to place, particularly under integral window sills, and it was necessary to shore the panels heavily to hold them

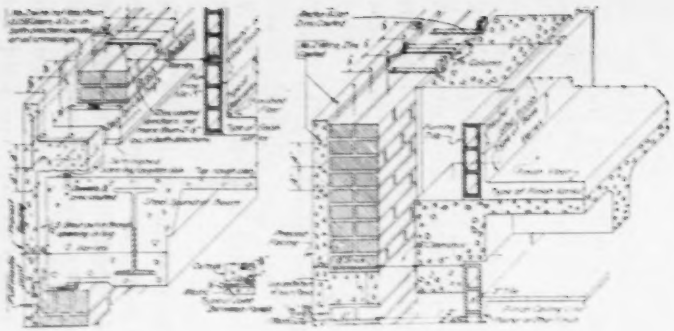


Fig. 10 (top—detail at left)—Detail showing original method of mounting precast panels on spandrel beams of steel structural framing—a method modified on the job (see "Design of Panels" and Fig. 11 and 12)

Fig. 11 (below)—Anchorage of setting lug of facing panels to spandrel beams by steel dowels in drill holes; later grouted in when gaps in lugs were filled

Fig. 13 (top—detail at right)—Detail of a typical setting of precast panels for walls of wings of the main building, ward building and other 3-story construction

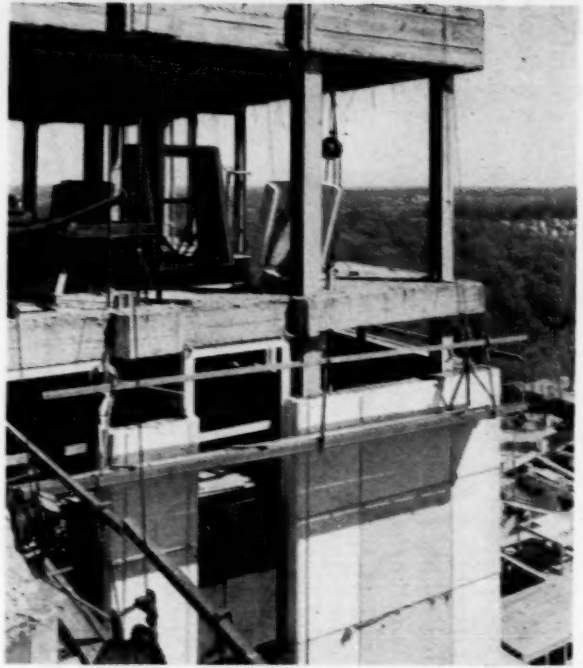


Fig. 12—Placing and backing of facing panels on the Tower

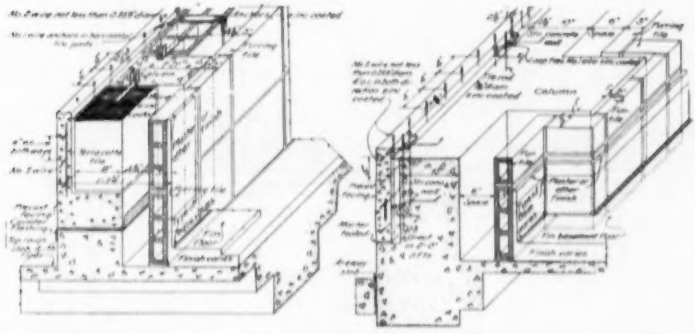


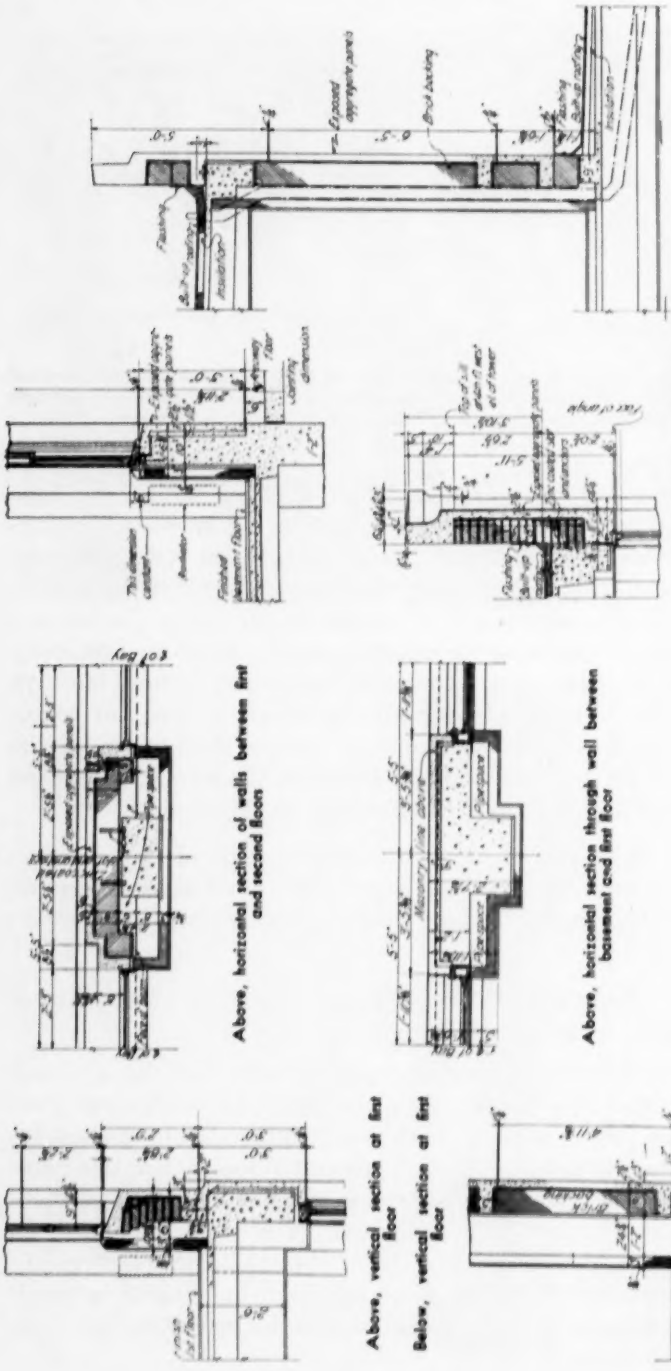
Fig. 14—At left typical wall section at fourth floor set-back of wings; at right, typical condition for basement walls as originally designed to secure precast panels to concrete walls of basements; modifications were used in actual construction

against the pressure of the grouting. This grouting had to be done very carefully, since there is no direct mechanical tie, and the effectiveness depends on complete filling of the space between rods and loops with good concrete. This method of construction has been used satisfactorily in mounting interior panels of plastic mosaic (as at Carderock), but is not considered desirable for exterior panels, which are generally much larger and with complicated returns, and where a more effective anchorage and seal is required to resist the effects of weather conditions. The panels should either have been used as face forms for the concrete walls, as at the Naval Model Basin at Carderock, or backed with brick or other masonry, as used elsewhere on this project.

Fig. 15 shows plans and sections through the walls of the wings on free standing construction. Fig. 16 shows the actual construction on walls of the ward buildings which are similar, although with some modifications.

Fig. 17 shows plans and sections through the walls of the Tower which are supported by structural frames.

It will be noted by the drawings and photographs that the architect has a greater freedom in placing joints as desired for architectural effect and that the cast panels allow a more nearly integral construction for watertightness than is practicable with natural stone. Also, the range of colors, by the use of various natural stones and manufactured ceramics and glass aggregates is practically unlimited. Fig. 18 of the anchor on the west face of the Tower and Fig. 19 of the medicinal plant panels over the west entrances of the wings of the main building show possibilities of ornamentation in this plastic medium at much less cost than carving of natural stone.



Left, above, vertical section at basement floor
Left, below, vertical section at roof
Above at right, vertical section of wall at fourth floor

Above, horizontal section of walls between first and second floors

Above, horizontal section through wall between basement and first floor

Above, vertical section at first floor

Below, vertical section at first floor

Fig. 15—Plans and sections through walls of the wings on free standing construction

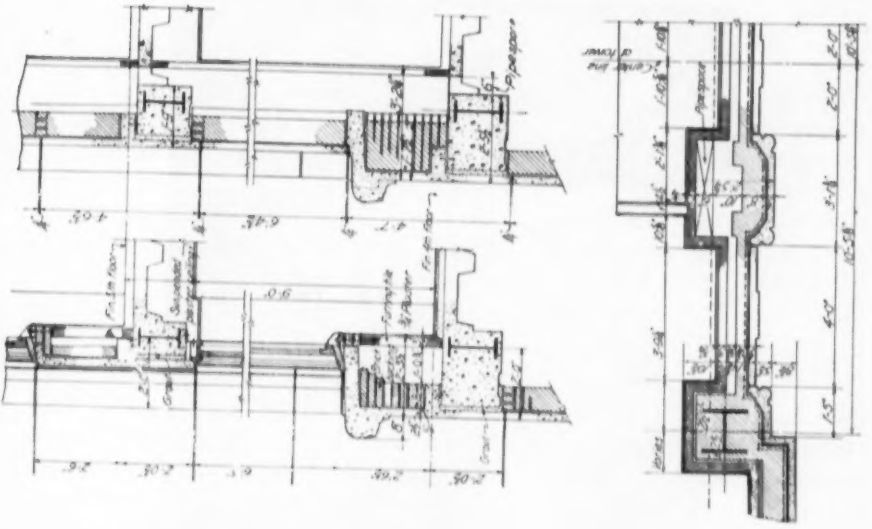


Fig. 16—Construction view of walls of ward buildings

Fig. 17—Plans and sections through walls of Tower supported on structural frame

Below, horizontal section above fourth floor
Right, above, vertical sections through end walls of tower, fourth and fifth floor
Right, below, horizontal section tower wall, ninth floor

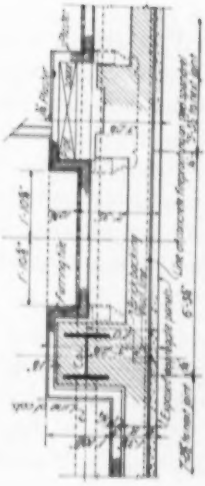


Fig. 18 and 19—Architectural ornamentation in exposed aggregate concrete

SETTING OF PANELS

The setting of the panels was done by an organization with extensive experience in setting natural stone on high-class building construction. Some difficulty was had at first in setting and holding the panels to the required degree of alignment, and the stone masons were at first unfavorably disposed to the new material. Later with more experience and better technique, it is understood these objections were overcome, but the fact remains that it is more difficult to set the large panels to good alignment than the smaller sections of natural stone for several reasons.

(1) It appears to be impossible to hold the dimensions, angles of returns and trueness of faces to the accuracy of well-seasoned and carefully machined natural stone. The panels as actually set had variations in dimensions and trueness of face of $\frac{1}{8}$ in. (and in some cases slightly more), which on close inspection definitely show up on this type of work.

(2) The area of the setting lug is smaller in relation to the face of the panel than the relative area of the setting bed of stone. Consequently, it was at times desirable to stay the upper edge of the panels until the brick masons had followed with the backing, and minor move-

ments sometimes occur, in cases requiring cutting out the brickwork and resetting the panel.

(3) Large panels, especially if they have deep returns, are more difficult to handle without damage, such as chipping edges. Fortunately field repairs can be made without showing, if done by qualified experts. This involves cutting a cavity similar to a dentist's preparation of a tooth for a filling, wetting the surface well, applying a thin coat of rich mortar worked in with a trowel, then filling the area with the exact mix used in casting the panel, water-curing properly, and finishing by rubbing and acid washing in the same manner as the original surface.

POINTING OF JOINTS

All joints were raked or cut out with air tools to a depth of an inch and pointed with a plastic caulking compound applied with an air pressure gun to insure watertightness. The joints of one course in each story of the tower were raked out the full depth of the panels and filled with caulking compound to give some freedom of movement to the panel system in following any sway of the tower, although very little movement is expected since the cruciform shape and heavily wind-braced design of the steel work should furnish a very rigid frame.

Caulking compounds

Since one of the major troubles with any building construction is water getting into the walls, an especial effort was made to obtain the best caulking compound available. Consultations were had with a representative of the Bureau of Standards, the Materials Engineer for the Procurement Division, and representatives of stone contractors for several of the finest buildings in Washington, to obtain their recommendations. They recommended a number of well-known brands of caulking compounds, but with the caution that they may be had in anywhere from one to four quality grades without very noticeable differentiation in labeling identification.

The Bureau of Standards had just completed extensive investigations to determine the lasting qualities of caulking compounds and had prepared a "Suggested Specification for Plastic Caulking Compounds" which appeared to be the best guide available. These specifications were accordingly incorporated in the specifications for the work at the Medical Center. Briefly, they set forth certain requirements and describe tests for shrinkage, bond, tenacity, rate of hardening, oil retention, and consistency.

The subcontractor for stone setting submitted samples furnished by five well-known producers of caulking compounds. On tests at the

Bureau of Standards all of them failed to meet the shrinkage requirement, and two failed to meet the rate of hardening requirement. However, one of these brands came very close to the requirements and was accordingly approved for use, subject to the actual lots received for the work on the job not exceeding the shrinkage of the test sample. Actually tests on the various shipments were slightly better and came just within the specification.

The subcontractor for caulking the windows submitted three successive samples made by one producer, each of which showed double the allowed percentage of shrinkage. He finally gave up and decided to use the brand approved for the stone contractor.

The subcontractor for the sheet-metal work had been authorized to use a caulking compound which was not among the recommended well-known brands for filling the inside of expansion joints, without submitting samples for test, since the service required was not of much importance. He also submitted a sample of a cartridged caulking compound of a brand not well known in this locality, for use in caulking flashing reglets. The latter is a very important service, and he was told that he might as well save the trouble of tests by using the brand approved for the panel work. He said, however, that the cartridges fitted his guns, and it would be more convenient for him to use the brand submitted if it passed the tests.

Accordingly a sample of the cartridged caulking compound and also a representative sample taken from the drum of material authorized for the inside of the expansion joints were sent to the Bureau of Standards for test. To our astonishment, after the previous difficulties with a number of well-known brands, both passed with flying colors, showing a shrinkage of only two-thirds and rates of hardening of only half to three-quarters of the allowed amounts.

IN CONCLUSION

It will be noted that extensive investigations and studies were made preliminary to the design of the panels and that every element of the work from the production of the panels at the plant, through handling and setting, to the final caulking of the joints, was carefully supervised and inspected. However, the work in place is far from perfect in detail. On close inspection, there is apparent some variation in surface texture and color, some misalignment of panels shows up particularly when the sun is nearly parallel to the surface, and some repair work is visible especially when the surfaces are wet.

The point to be emphasized at all times is that the production of architectural concrete is an art as well as a scientific process and that

a high degree of skilled and conscientious craftsmanship is essential to every operation. These craftsmen are not developed overnight.

Another factor is time. As the poet said, "Art is long" but in modern building construction the contractor is constantly under pressure to get the building closed in so that other work can proceed, and the architect or engineer is under pressure to accept sub-standard work to get the building ready for service. The moral is that the producer's plant should have a normal capacity of at least double the weekly output called for by the setting schedules, particularly as, at least in the earlier stages, a considerable proportion of the panels are not economically produced in the sequence called for by the erector.

Despite the difficulties and defects in detail above mentioned, the work as a whole at the Medical Center is considered very satisfactory. The design of the panels carries out the architectural conception in excellent manner. The surface has light and color and a distinctive character of its own which is not an imitation of any natural stone. The effect as a whole can best be appreciated by an actual visit, which Institute members are cordially invited to make.

Discussion of this paper should reach the ACI Secretary in triplicate by June 1, 1942, for publication in the JOURNAL Supplement, concluding Proceedings V. 38.

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Proposed Specification for Cast Stone*

Reported by ACI Committee 704

C. G. WALKER
Chairman

M. A. ARNOLD
OTTO BUEHNER
LOUIS GUZZETI
N. D. HARTER
JOHN TUCKER, JR.

The following proposed specification for cast stone is intended to supercede the specification tentatively adopted by the American Concrete Institute in February, 1929 (P3-A-29T) and which has since been generally recognized and used commercially as a standard for the physical quality of cast stone. The principal changes which have been made are an increase in compressive strength requirements from 5,000 psi. to 6,500 psi. and a reduction in permissible absorption for 7 per cent to 6 per cent. The absorption period has been increased from 24 to 48 hours, and the lower limit of 3 per cent on absorption has been eliminated.

I. GENERAL

1. The term "cast stone" shall be understood to mean a building stone manufactured from portland cement concrete, precast and set in place as trim or facing on or in buildings and other structures.

2. The minimum compressive strength of cast stone when delivered to the building site shall be 6,500 psi. when tested as 2 by 2-in. cylinders or 2 by 2-in. cubes in the manner hereinafter specified.

3. The average water absorption of cast stone when delivered to the building site shall be not more than 6 per cent by dry weight of the specimens when tested as 2 by 2-in. cylinders or 2 by 2-in. cubes in the manner hereinafter specified.

4. All aggregate used in the manufacture of cast stone shall be of known durable quality.

*Released by the Standards Committee for publication and discussion as approved by all members of Committee 704.

II. SELECTION OF SPECIMENS FOR TESTING

5. Specimens for both compression and absorption tests shall be cut from stone as delivered to the job or from regular stock in the yard. Samples from which specimens will be cut shall be selected by the purchaser or his representative.

6. Specimens of faced cast stone for compression tests shall be cut in such a manner that the specimens are composed of approximately one-half of facing and one-half of backing material. Specimens shall be tested in the position in which the cast stone is laid in the wall.

III. METHODS OF TESTING

7. Not less than 3 and preferably 5 specimens shall be required for each test.

8. Specimens for absorption test shall be dried at a temperature between 215 and 225 F. until the loss in weight is not more than 0.1 per cent in 24 hours of drying. They shall be allowed to cool to room temperature, weighted and then submerged in water at a temperature between 60 and 80 F. for 48 hours. The specimens shall then be removed from the water, the surface water wiped off with a damp cloth and the specimens weighed. The percentage of absorption is the difference in weight divided by the dry weight of the specimen and multiplied by 100.

9. Specimens for strength test shall be dried at a temperature between 215 and 225 F. until the loss in weight is not more than 0.1 per cent in 24 hours of drying.

10. If bearing surfaces of specimens for strength tests are not smooth, they shall be made so by grinding. If they cannot be ground to a smooth surface they shall be capped with a mixture of ½ part portland cement and ½ part plaster of paris, which shall be allowed to harden at least 5 hours before the test. The cap shall be formed by spreading the capping material upon a plate glass and pressing the specimen firmly on it making the cap as thin as possible. On specimens that are to be capped, all surfaces which are likely to come into contact with the capping material shall be shellacked and then allowed to dry before the capping is applied.

11. Load shall be applied through a spherical bearing block placed on top of the specimen in a vertical testing machine. The dimensions of the bearing block shall be the same, or slightly larger, than those of the test specimen.

12. Load shall be applied uniformly and without shock. The speed of the moving head of the machine shall be not more than .05 in. per min. when the machine is running idle.

13. Specimens shall be loaded to failure and the unit compressive strength calculated in pounds per square inch. The type of failure and appearance of specimen shall be noted.

Discussion of this report should reach the ACI Secretary in triplicate by June 1, 1942, for publication in the JOURNAL Supplement, concluding Proceedings V. 38.

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Discussion of report of Committee 704:

Proposed Specification for Cast Stone*

CONVENTION DISCUSSION BY C. G. WALKER, CHAIRMAN COMMITTEE 704, SUPPLEMENTING THE PROPOSED SPECIFICATION†

TESTS OF INTEGRAL AND SURFACE WATERPROOFINGS

In connection with the proposed specification requirement that the 48-hour absorption of cast stone when delivered to the building site shall not be more than 6 per cent, I should like to present some results from tests on the use of integral and surface waterproofings as a means of reducing absorption. In spite of the volumes of written and spoken discussion during the past 20 years on the subject of waterproofings, there still seems to be about as much disagreement as ever and there are still few unprejudiced laboratory data on the effectiveness of waterproofing in keeping moisture out of concrete.

In order that you may understand why the Cast Stone Institute, which I represent, is so concerned with the matter of absorption, I would like to remind you that cast stone is primarily a decorative material. It must have and must retain a pleasing appearance in order to be satisfactory. In other words, its face is its fortune. Its face must not become rough or discolored with age. It must retain its good looks throughout the life of the structure of which it is a part. The surface of an inferior, highly absorptive cast stone becomes etched and pocked by the dissolving or leeching out of part of the cementing material between the aggregate particles. The freezing of water absorbed into the surface of inferior cast stone and the alternate expansion and contraction due to the absorption and loss of moisture cause minute cracks to be formed which first appear as tiny wrinkles but which eventually develop into serious disfigurement. Collectively, these factors produce what is known as weathering.

*ACI JOURNAL, Feb. 1942; *Proceedings* V. 38, p. 313.
†Published in advance of the announced schedule to be considered with the "Proposed Specification for Cast Stone" for the discusson of which the closure date has been postponed to July 15.

Most cast stone manufacturers are agreed that erosion and other weathering effects of moisture penetration into the surface of cast stone are responsible for some of the most serious criticism that they encounter. It was as a means of obtaining some unprejudiced information on the possible effectiveness of various waterproofings in preserving the good appearance of cast stone that the Cast Stone Institute began a series of laboratory tests in 1938, using facilities generously provided by the Research Laboratory of the Portland Cement Association.

Two-inch molded cubes in sets of three were used as the principal test specimens. They were made with a representative cast stone mix—1 part of cement to 3½ parts of dry marble aggregate by weight with 6¾ gal. of mixing water per sack of cement. The aggregate was uniformly graded with about 4 per cent retained on the No. 4 sieve and about 4 per cent passing the No. 100 sieve. At 28 days of age, the compressive strength of three unwaterproofed specimens averaged 5,890 psi and the absorption was 7.5 per cent. Specimens were cured 5 days in a fog room after which the specimens to which surface waterproofing was applied were allowed to dry in the air of the laboratory for 5 days before being treated. Twenty-eight types or conditions of waterproofing were covered. Integral waterproofings were all of the water repellant type. Seven were paste or liquid and eight were powder. The powdered waterproofings were ground into the cement by mortar and pestle before it was mixed with the aggregate. Surface waterproofings were of the colorless type. They were applied in two coats about 4 hours apart.

At the age of 11 days, all specimens were placed outdoors where they were continuously under exposure to the weather during the time between tests. At 73 days of age and at variable intervals thereafter the specimens were tested for absorption under the standard procedure for making absorption tests, determinations being made on absorption after immersion for periods of 1, 3, 6, 24 and 48 hours and after boiling for 5 hours. This method of testing the effectiveness of waterproofings was used because in cast stone we are concerned primarily with absorption through capillary attraction. We do not have to worry much about permeability. Any cast stone that is worthy of the name is impermeable, although it must be admitted that once in a while we run across examples of so-called cast stone through which water can literally be poured. It is also because we are concerned with absorption through capillary attraction that our tests were made chiefly with waterproofings of the water repellant type rather than of the pore-filling or surface hardening type.

For the benefit of those who are interested in making their own interpretations of test data, some of the results from these tests have been

made available together with information on the waterproofing materials. (Tables A and B). One of the most noticeable things about these results is the relatively high absorption shown in the first test and the marked decrease as shown in successive tests. The reduction comes, of course, from the curing that normally accompanies aging, accelerated in this case by the repeated soaking and boiling to which the specimens were subjected. Some specimens show a smaller decrease in absorption than others and a few show an increase because the improvement due to age and curing was offset by loss of effectiveness in the waterproofing. It is customary, of course, to expect and even to count upon an improvement of concrete with age. However, in the case of cast stone there is plenty of reason to believe that under ordinary conditions actual or at least incipient damage to appearance takes place long before the material's potential resistance to weathering has had time to develop. If that is so, then it is clear that some sort of treatment which will give at least temporary protection to the surface of newly erected cast stone is essential for satisfactory results.

Another very noticeable point is that very few of the waterproofings used in these tests were capable of reducing 48-hour absorption to 6 per cent. However, this should be considered in the light of the amount of mixing water used, which was 6¾ gal. per sack of cement. Supplementary laboratory tests show very definitely that integral waterproofing is considerably more effective in mixes of lower water-cement ratios and that a 48-hour absorption of 6 per cent or even less is readily obtainable through simple adjustments of the mix. Tests on commercial cast stones substantiate this. In view of this and of the outstanding results produced by a few of the waterproofings in this test, even in a 6¾ gal. mix, we of the Cast Stone Institute believe that the proposed limitation on absorption is commercially practicable. We believe also that the possibility and practicability of making cast stone with much less than the present limitation of 7 per cent on absorption is to be regarded as an improvement in the art and that it should be incorporated in the standard of quality for cast stone.

Still another point worth noting in these results is the effect of different waterproofings on rate of absorption. Several of the materials which did not show up well under 48 hours of immersion or under boiling did a very efficient job of keeping the water out for several hours. Considering their probable effectiveness under conditions of ordinary use and exposure, many of the materials included in these tests would undoubtedly be more beneficial than is indicated by the results under the standard procedure of making absorption tests. In connection with this matter

TABLE A—ABSORPTION TESTS WITH INTEGRAL WATERPROOFINGS

Results are from tests repeated on same specimens at ages shown.

(Tests were also made at 166, 257 and 809 days of age.) All values are the average of at least three specimens.

Ref. No.	Age at test; days	Absorption for period indicated—per cent 1 Hr.	3 Hr.	6 Hr.	24 Hr.	48 Hr.	Boil	Waterproofing % by wt. of cement
X	73	5.78	6.80	6.91	7.08	7.16	8.02	None
	371	2.25	3.08	3.86	6.03	6.60	7.39	
	1170	1.59	1.90	2.20	3.60	5.05	7.38	
A	73	4.59	6.65	7.26	7.49	7.58	8.48	¼% calcium stearate; Brand "A"
	371	2.06	2.64	3.26	5.59	6.69	7.53	
	1170	1.72	1.75	2.23	3.48	4.86	8.02	
B	73	1.97	2.87	3.85	6.57	7.08	8.14	½% calcium stearate; Brand "A"
	371	1.46	1.73	1.95	2.76	3.56	8.18	
	1170	1.46	1.67	1.83	2.56	3.47	7.56	
C	73	1.36	1.54	1.74	2.50	3.08	13.65	½% calcium alum. stearate; Brand "A"
	371	2.20	2.74	3.17	4.29	5.09	12.58	
	1170	2.49	3.13	3.54	4.81	5.72	11.60	
D	73	2.46	3.58	4.69	6.99	7.25	8.68	½% calcium stearate; Brand "B"
	371	1.85	2.31	2.79	4.64	5.84	8.27	
	1170	1.71	1.95	2.18	3.29	4.59	8.47	
E	73	4.52	6.60	7.22	7.46	7.56	8.36	¼% calcium stearate; Brand "C"
	371	2.03	2.73	3.46	5.93	6.61	7.45	
	1170	1.55	1.81	2.05	3.25	4.62	8.12	
F	73	2.13	3,27	4.39	7.12	7.47	8.31	½% calcium stearate; Brand "C"
	371	1.65	2.03	2.41	5.02	5.19	8.08	
	1170	1.54	1.79	1.99	3.03	4.28	8.09	
G	73	2.92	3.97	4.89	6.39	6.68	8.36	½% aluminum stearate; Brand "C"
	371	1.62	1.89	2.13	3.00	3.84	8.46	
	1170	1.54	1.77	1.89	2.49	3.22	8.03	
H	73	3.32	4.89	5.93	6.66	6.77	7.61	1% ammonium stearate; Brand "B"
	371	1.90	2.53	3.20	5.41	6.24	7.10	
	1170	1.46	1.75	1.86	2.97	4.27	8.00	
I	73	2.15	2.94	3.65	5.91	6.65	8.71	2% ammonium stearate; Brand "B"
	371	1.66	1.98	2.28	3.52	4.61	8.55	
	1170	1.60	1.92	2.11	3.42	4.84	8.08	
J	73	2.72	3.99	5.17	7.24	7.53	10.51	1% trimethylamine stearate; Brand "D"
	371	2.13	2.46	2.80	4.10	5.33	10.87	
	1170	2.06	2.40	2.64	3.91	5.29	10.65	
K	73	1.06	1.37	1.60	2.62	3.52	13.09	2% trimethylamine stearate; Brand "D"
	371	1.07	1.30	1.45	2.05	2.47	11.95	
	1170	1.41	1.99	2.51	4.36	5.35	10.92	
L	73	.85	1.07	1.23	1.84	2.32	5.75	1% butyl stearate emul. Brand "D"
	371	.89	1.02	1.10	1.51	1.68	6.01	
	1170	1.06	1.31	1.52	2.19	2.88	6.07	
M	73	.76	.88	.98	1.32	1.58	3.65	2% butyl stearate emul. Brand "D"
	371	.66	.78	.85	1.05	1.42	3.45	
	1170	.75	.84	.90	1.28	1.66	5.41	

TABLE B—ABSORPTION TESTS WITH SURFACE WATERPROOFINGS

Results are from tests repeated on same specimens at ages as shown.

(Tests were also made at 166, 257 and 809 days of age.) All values are the average of at least three specimens. Surface waterproofings used in these tests included nine proprietary materials and laboratory preparations as follows: potassium silicate mixture; aroclor in toluol, paraffin in carbon tetrachloride, Mag-Zin fluosilicate mixture; butyl stearate, soya bean oil in turpentine.

Ref. No.	Age at test; days	Absorption for period indicated—per cent					
		1 Hr.	3 Hr.	6 Hr.	24 Hr.	48 Hr.	Boil
Unwaterproofed	73	5.78	6.80	6.91	7.08	7.16	8.02
	371	2.25	3.08	3.86	6.03	6.60	7.39
	1170	1.59	1.90	2.20	3.60	5.05	7.38
AA	73	5.56	6.98	7.06	7.20	7.26	8.01
	371	1.77	2.20	2.60	4.37	5.78	7.56
	1170	1.58	1.77	1.97	3.08	4.44	7.49
BB	73	6.52	7.06	7.12	7.27	7.34	8.03
	371	2.21	3.07	3.95	7.41	6.91	7.46
	1170	1.66	2.00	2.32	3.92	5.50	7.30
CC	73	5.68	7.08	7.16	7.34	7.39	7.84
	371	2.56	3.55	4.75	5.82	7.07	7.54
	1170	1.62	1.99	2.37	4.18	5.93	7.39
DD	73	3.99	6.85	7.06	7.29	7.34	7.96
	371	2.44	3.59	4.74	6.65	6.94	7.66
	1170	1.81	2.20	2.60	4.53	6.08	7.57
EE	73	.40	.64	.86	1.75	2.53	7.84
	371	1.37	1.63	1.85	2.88	3.65	8.20
	1170	1.54	1.85	2.14	3.83	6.01	7.05
FF	73	6.43	7.11	7.22	7.39	7.47	8.27
	371	3.20	4.87	6.21	7.22	7.30	7.97
	1170	1.70	2.01	2.37	4.14	5.83	7.32
GG	73	.22	.41	.64	1.75	2.94	7.85
	371	1.33	2.23	3.08	——	6.64	7.33
	1170	1.60	2.09	2.57	5.10	5.96	6.93
HH	73	6.17	6.70	6.75	6.93	7.00	7.68
	371	3.81	5.88	6.61	6.87	6.93	7.55
	1170	1.83	2.43	2.96	4.46	5.46	7.30
II	73	1.64	2.41	3.48	6.22	6.78	7.74
	371	1.33	1.49	1.64	2.29	2.49	7.91
	1170	1.26	1.43	1.64	2.60	3.73	7.01
JJ	73	4.04	6.74	7.06	7.28	7.36	7.70
	371	2.24	3.79	5.47	7.15	7.22	7.51
	1170	1.51	1.83	2.15	3.59	5.12	7.35
KK	73	4.76	6.95	7.12	7.31	7.36	8.03
	371	1.69	2.20	2.69	4.66	5.91	7.57
	1170	1.27	1.49	1.68	2.76	4.12	7.70
LL	73	.15	.23	.34	.54	.79	2.75
	371	.26	.33	.36	.66	.90	4.96
	1170	.46	.53	.60	.94	1.38	6.24
MM	73	5.79	6.99	7.15	7.19	7.39	7.85
	371	2.69	4.27	5.76	7.00	7.08	7.47
	1170	1.54	1.91	2.27	3.96	5.77	7.04
NN	73	.34	.50	.65	1.16	1.64	8.06
	371	.89	1.24	1.40	2.13	2.83	8.11
	1170	1.59	1.93	2.16	3.27	4.29	7.00

of rate of absorption, the suggestion has been made that a requirement on rate of absorption be incorporated into the proposed specification. It seems entirely logical that the speed with which moisture can penetrate cast stone is as important as the amount that can ultimately get in and I personally would be in favor of a provision on rate of absorption.

There is another point which needs to be kept in mind in the use of integral waterproofings in concrete. A great many people in the concrete field appear to have an idea that waterproofing aids curing by keeping concrete from drying out as fast as it would otherwise. No evidence supporting such a belief was found in these tests. During the five days of drying in the air of the laboratory the average loss of moisture in 54 cubes containing no waterproofing was 4.24 per cent of the original weight, the lowest being 3.28 per cent and the highest 4.98 per cent. The loss of moisture in thirteen different groups of integrally waterproofed specimens ranged from 3.62 per cent to 5.75 per cent. Instead of being an aid to curing, a really effective waterproofing may in fact be a decided hindrance by preventing the entrance of moisture to replace that which is lost through evaporation. For that reason, it would seem highly advisable that waterproofed concrete should be kept continuously and completely moist until it has developed the desired physical quality. Intermittent spraying will not restore the moisture lost by evaporation between sprayings.

A comparison of the results from these tests indicates that from the standpoint of keeping water out of concrete, the integral waterproofings as a group were more effective than the surface waterproofings as a group. There were, of course, exceptions in both groups. However, it should be mentioned that the specimens which showed the least effect of weathering were among those treated with surface waterproofing. On the whole, the specimens showing the lowest absorptions were the least darkened by exposure and they were markedly faster in drying out after being wet. None of the waterproofings affected the color of the specimens but some of the integral waterproofings had a marked aerating effect on the concrete and produced a cellular structure in the specimens which caused the surface to catch and hold dirt noticeably more than others.

In general, it must be admitted that results from these tests confirm the conclusions reached by other investigators; namely, that the performance of all too many of the so-called waterproofings does not come up to the claims made for them. At the same time, it is clearly indicated that by the careful selection and proper use of either an integral or sur-

face waterproofing, moisture penetration through capillary attraction can be greatly reduced. That is the point which we are concerned with in considering a specification requirement on the absorption of cast stone.

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Concluding Discussion of report of Committee 704:

Proposed Specification for Cast Stone*

DISCUSSION BY JOHN TUCKER, JR., A. E. HORN,
C. G. WALKER, Chairman

By JOHN TUCKER, Jr.†

Mr. Walker presents absorption data which are of considerable interest. It cannot be emphasized too strongly that the integral "waterproofings" in his tests are of a wide range of effectiveness. The term "waterproofing" in these tests is misleading, water repellancy being the property measured. Although all of the "waterproofings" were effective in the 1 hr. tests, only 4 of the 13 were markedly effective in the 48 hr. test, and only two (L & M) reduced the absorption upon boiling the specimen.

The absorption characteristics of a Cast Stone are of no direct interest, but are of importance in their indication of other importance characteristics of the stone. The absorption characteristics are related, sometimes more vaguely than desirable, to desirable properties of the stone, such as relative freedom from soiling and from crazing, and of the durability of the stone when it is exposed to the action of the weather. We should also note that maintenance of appearance (non soiling) is as yet not connected with resistance to weathering. For example, a material such as some marbles will "sand" slowly, and will present a continued clean surface, yet the material is surely weathering.

Mr. Walker has not brought out in detail to what extent the several integral "waterproofings" improved the stone in these three categories and it is suggested that he do so.

Mr. Walker has informed me orally that emulsified buytl stearate as admix maintained the cast stone in a very clean condition, and free from crazing, and that resistance to freezing and thawing was also greatly

*ACI JOURNAL, Feb. 1942; *Proceedings* V. 38, p. 313, and discussion Apr. 1942, p. 316 - 1.
†National Bureau of Standards, Washington, D. C.

increased. We doubt whether most of the other integral "waterproofing" were in any way as effective. The action of the butyl stearate was so excellent that consideration should be given to making its use mandatory.

The proposed specification stipulates a maximum absorption of 6 per cent (48 hr.). Because of the wide variation in the 48 hr. absorption of the various mixes, untreated, which would be encountered in commercial cast stone, a fact recognized by Mr. Walker, many samples would have absorptions less than 6 per cent. The indications are however, that the improved characteristics of the cast stone samples, was due not to the low absorptions themselves, but some property injected into the material by the use of the integral "waterproofing." Since the most effective material resulted in a 48 hr. absorption of less than 2 per cent, a limit of 6 per cent on the absorption appears too high and the specification should require that the absorption be well under that figure.

The ratio of the amount of water absorbed by a specimen in a short time, compared to the amount absorbed by boiling (the Krüger or Schurecht Ratio) or the saturation coefficient, as determined by Stull and Johnson, N.B.S. Research Paper 1349, are the best measures of the resistance of building materials to destructive weathering. Such a criterion is incorporated in the Federal Specification for Cast Stone, and should, we believe, be incorporated in the ACI specification.

In closing, we would congratulate Mr. Walker on his efforts in developing the specification.

By A. E. HORN*

The tests on integral waterproofings are specific for materials indicated, but are by no means comparable.

In the series of tests running from X co M the percentages run the gamut of ammonium stearate, having an anhydrous ammonium stearate content of 13 per cent, to aluminum stearate of 100 per cent anhydrous aluminum stearate content and butyl stearate of a probable 50 per cent anhydrous content.

The probable value lies in the stearate radicle—

Calcium stearate contains	93.2 per cent stearate
Aluminum stearate contains	96.8 per cent stearate
Ammonium stearate contains	94.6 per cent stearate
Butyl stearate contains	83.1 per cent stearate

as well as the ready miscibility of these stearates with the distributing agency, i. e. the water.

For this purpose the selection of the cheapest soluble or miscible stearates is desirable. To use 2 per cent of butyl stearate emulsion, which

*A. C. Horn Co., Long Island City, N. Y.

probably contains 50 per cent of water, means that 1 per cent of butyl stearate was used which contained 0.831 lb. of stearic acid. To use 1 per cent ammonium stearate, which contains 87 per cent water, means that 0.13 of 1 per cent of ammonium stearate was used—less than the actual butyl stearate used and compared to the stearic acid content only 0.124 of 1 per cent was utilized.

To make the tests equivalent, approximately seven times the amount of ammonium stearate should have been used, or 1/7th as much of butyl stearate.

The ready miscibility and distribution of the compounds should likewise be considered. Ammonium stearate is the only stearate soluble in water, whereas the others—with the exception of butyl stearate—are with difficulty miscible with water.

CLOSURE BY C. G. WALKER, Chairman Committee 704

In his discussion of the proposed specification for cast stone, Mr. Tucker suggests that the limit on the 48-hour absorption might well be set lower than 6 per cent. While data submitted in connection with the report of Committee 704 show that 48-hour absorptions of considerably less than 6 per cent are obtainable, it is questionable whether that warrants the arbitrary establishment of a lower limit at this time. This is especially true if we consider that the purpose of the proposed specification is not so much to set up requirements insuring the ultimate of physical quality in cast stone as to provide a yardstick of commercial quality for the guidance of users. With that in mind, it is to be hoped that the proposed specification can be adopted as an Institute standard at its next convention.

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Good Practice in Concrete Masonry Wall Construction*

BY KENNETH C. TIPPY†

SYNOPSIS

The author discusses some of the less understood but important details of concrete masonry wall construction details which represent the difference between ordinary and good construction—strong, durable and watertight; the need for preshrinkage of moisture-laden masonry units before laying in a wall; the use of the proper mortar; adequate footings and foundations; use of proper lintels and sills; partitions tied to exterior walls; expansion and contraction joint; precautions with parapet and flashings and with drains and waterproofing practice.

The purpose of this paper is to discuss some of the less understood but nevertheless important details which make the difference between ordinary and good construction with concrete masonry. General details are available in several publications issued by the National Concrete Masonry Association and the Portland Cement Association. Moreover, they are widely accepted as standard practice in the construction industry. There are, however, certain details which, if not properly handled, may lead to unsatisfactory results and it is these that will receive attention herein.

The main problem in concrete masonry construction is to build strong, durable, watertight walls. That concrete masonry is an excellent building material is evidenced by the many satisfactory buildings which have been built with concrete units as the sole or principal structural wall material. In the main these buildings have not cracked or leaked, or such failures as have occurred have been of a limited nature. A few buildings have cracked or permitted leakage. Such failures as we have investigated, however, have indicated sub-standard masonry units, poor workmanship or faulty design. No building material available today is proof against these faults.

*Received by the Institute Jan. 7, 1942.
†Cement Products Bureau, Portland Cement Assn., Chicago.

REDUCING WALL SHRINKAGE

All manufactured masonry materials shrink during the process of manufacture: clay products while in the drying room and in the kiln; concrete products during the curing and drying out period. This shrinkage is a definite part of the manufacturing process and neither material should be considered a finished product until a large part of this shrinkage has taken place. Concrete masonry units laid in a wall in a green or wet state will later dry out and the resulting shrinkage is frequently sufficient to cause cracking.

Studies made in the Portland Cement Association laboratories and reported by the Institute* showed that the moisture content of concrete masonry units at the time they are laid in the wall has an important bearing on the extent of shrinkage. These studies included tests of 24 panels of concrete masonry units made with three different aggregates. The following example is typical of the panels tested:

Panels erected with saturated two day old units containing 19.3 per cent moisture by weight contracted .54 in. per 100 ft. in 45 days. Further shrinkage due to the heating of the building in which the panels were housed resulted in a total shrinkage at 150 days of about .67 in. per 100 ft.

Panels erected with seven day old units containing 7.3 per cent moisture contracted about .35 in. per 100 ft. in 30 days. Total contraction after the building was heated was about .50 in. at 150 days.

28 day old units when erected in panels contained 4.9 per cent moisture. Total shrinkage for 150 days was 0.35 in. per 100 ft.

Panels erected with 90 day old units containing 2.8 per cent moisture expanded slightly during the first two days before contraction started. Total shrinkage was about 0.17 in. per 100 ft.

These studies showed that air drying of the units for 5 days reduced the shrinkage for the 150 day period to about 75 per cent of that obtained with saturated units. Similarly, 26 days of air drying reduced the shrinkage to about half that obtained with saturated units. Eighty-eight days of air drying reduced the shrinkage to about 26 per cent of that of the saturated units.

Present Federal and A. S. T. M. Specifications limit the moisture content of concrete masonry units at the time of delivery to the job to 40 per cent of the total possible absorption and this figure is generally accepted by others. This is something more than twice as much moisture as is contained in fully air-dry units. Tests on forced air drying of concrete masonry units conducted by the Portland Cement Association have shown that these units are in a fully air-dry condition when

*"Some Tests of Concrete Masonry Units Cured with High Pressure Steam," P. M. Woodworth; ACI JOURNAL, Feb., 1930; *Proceedings* V. 26, p. 504.

they contain from 15 to 25 per cent of their total absorption and that when they are dried further than this they take up moisture from the air.

From these data it is evident that concrete masonry units should be in as near an air-dry condition as possible prior to being used. Certainly they should never have more than 40 per cent moisture. Unfortunately, however, too many products manufacturers send out green, uncured or wet units and trouble frequently results.

In general concrete masonry units should be at least 28 days old before being placed in the wall. After the wet curing period they should be stockpiled so that the effect of wind, rain and sun can be regulated as required. Responsibility does not cease with the delivery of the block. They should be stacked on the job free from contact with the ground and should be covered with a tarpaulin or building paper to keep off the rain. The masons should cover the top of the unfinished wall with waterproof paper upon leaving the job at night or when it rains. These precautions will pay good dividends.

MORTAR FOR CONCRETE MASONRY

There are several types of mortar, containing portland cement, available for masonry wall construction. Some of these are factory mixed and consist of a mixture of portland cement, hydrated lime and waterproofing compounds. The proportions of these ingredients vary with the manufacturer but usually approximately equal volumes of portland cement and lime are used. Job mixed mortars are also used to a considerable extent. The proportions of portland cement, lime and sand for job mixed mortars are commonly 1:0:.25:3; 1:1:6; 1:2:9. The lime may be hydrated lime, hydrated lime putty or slaked lime putty.

The recommendations of the Portland Cement Association for mortars for masonry walls are as follows:

1. For severe exposure; below-grade construction; tornado, hurricane or earthquake regions; cold weather construction; or where higher than normal building code stresses are used: One volume of portland cement and not more than three volumes damp, loose mortar sand to which approved amounts of plasticizing agents may be added.

2. For all exterior walls, fire walls, party walls and chimneys, and for walls of hollow masonry units other than provided for in Section 1: Mortar specified in Section 1; or, one volume of portland cement, not more than one volume of lime putty or hydrated lime, and damp, loose mortar sand equal to not more than three times the combined volumes of cement and lime; or, one volume of masonry cement meeting the requirements for Type II of Federal Master Specifications SS-C-18lb., and not more than three volumes of damp, loose mortar sand.

Durability and resistance to frost action has been recognized in standard specifications as one of the important properties of a masonry unit. Similarly, the mortars which may comprise 25 per cent by volume of the masonry unit in the wall and 20 per cent of the exposed surface should have adequate frost resistance. The mortars recommended will have greater frost resistance than mortars leaner in portland cement content.

These recommendations for mortar mixes are based on the use of a well graded mortar sand with the requirement that 95 to 100 per cent pass a No. 8 sieve, 35 to 70 per cent pass a No. 30 sieve, 15 to 35 per cent pass a No. 50 sieve and 0 to 15 per cent pass the No. 100 sieve. Furthermore, the sand should be so graded that the proportion finer than a No. 16 sieve and coarser than a No. 30 sieve or that finer than a No. 30 sieve and coarser than a No. 50 sieve does not exceed 50 per cent. The use of very fine mortar sands will result in weaker mortars because of higher water requirements for a satisfactory plasticity. Use of richer mixes to overcome deficiency in sand grading and to meet strength requirements is not desirable due to increase in volume change of the mortar.

FOOTINGS AND FOUNDATIONS

Adequate footings and foundations are an essential requirement for any masonry wall construction. The size of footing depends upon the load to be carried and the safe bearing value of the soil. Where footings are supported on soil, some settlement always occurs and it is important so to proportion the widths of such footings that this settlement will be uniform. Too often, particularly in residential construction, all footings are made the same width regardless of the load to be carried. The fact that interior load-bearing partitions may carry up to twice the load of exterior wall footings is often overlooked.

SILLS AND LINTELS

Reinforced concrete lintels for windows and doors should be well bonded into the masonry on either side of the opening. They should have a bearing of at least 8 in. and the bearing should be increased for lintels of long span or exceptionally heavy loads. Possibly a good rule would be to provide a bearing length equal to the depth of the lintel. Provision of suitable bearings for lintels helps to prevent cracking at the corners of such openings.

Lintels may be made in one piece or they may be made in two pieces, the latter type being known as a split lintel. The reasons for making split lintels are two-fold—they are lighter and therefore easier to handle and the air space between affords insulation which is desirable especially if furring is not provided. Split lintels are quite satisfactory and are even desirable when they support wall loads only. However, when joists or beams bear on the lintel, it should be made in one piece because it is practically impossible to design the inner section of a split lintel, carrying both wall and floor loads, to have the same deflection as the outer section which carries wall load only. Differences in deflection of the two pieces would probably result in cracks in the masonry wall

above. Lintels more than 8 ft. long should be checked for deflection and when necessary the depth of the lintel should be increased.

There are two general types of sills in common use—the slip sill and the lug sill. Both types are sloped on the top face to drain water away quickly. They should project at least 1½ in. beyond wall face and be provided with a groove along the lower outer edge to provide a drip.

Slip sills are popular because they can be inserted after the wall proper has been built and therefore require no protection during construction. Since there is an exposed joint at each end of the sill, special care should be taken to fill it completely with mortar, packing the joints tight.

Lug sills are so called because the ends project into the masonry wall. There are no vertical joints at the juncture of the sills and jambs which is one of the advantages of the lug sill over the slip sill. Frequently they are made with washes at either end to divert water away from the jambs and this is a desirable feature.

At the time lug sills are set, only the portion projecting into the wall is bedded in mortar. The portion immediately below the wall opening is left free of contact with the wall below. This is done so that in case there is a minor settlement or adjustment in the masonry work during construction, the sill will be freer to adapt itself to such minor wall movements, thus avoiding possible damage to the sill during the construction period. The space between the sill and the wall below is filled with mortar and pointed after the exterior walls have been built.

WALL REINFORCEMENT

Some concrete products manufacturers have advocated the use of reinforcement in some of the horizontal mortar joints of concrete masonry walls as a means of preventing cracking. Some of these manufacturers have used such reinforcement for a period of several years and claim it has been effective in preventing cracks.

There are no data available, however, to prove this contention and such statements are based on the experience of individuals with certain jobs. Some observers think it is of no particular value; others seem to think that this type of reinforcement distributes stresses and controls cracking to the extent that cracks are finer and more widely distributed.

Round pencil rods (¼-in.) placed in each face shell mortar joint of the courses so reinforced is generally used, and in the past year a commercial type of reinforcement consisting of two longitudinal wires welded to diagonal cross wires has been developed. In so far as such reinforcement increases the tensile strength of the wall, it should help to prevent the formation of wide cracks at critical sections. It should never be regarded as a substitute for proper curing and drying of the

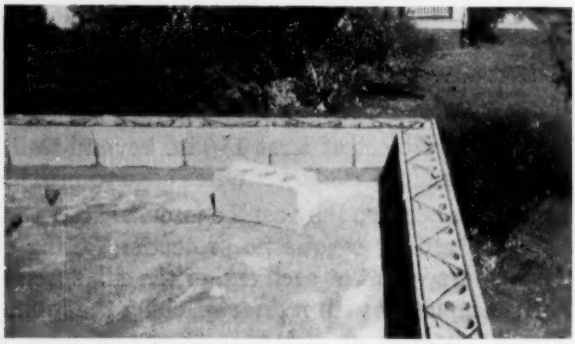

Fig. 1—Reinforcement in a masonry wall; questionable practice in a national emergency of steel shortage

units or as a "cure-all" for poor design and construction. In the light of our present knowledge, such reinforcement should not be regarded as anything more than a supplementary aid in controlling shrinkage cracks.

BONDING WALLS

Interior partitions are tied to the exterior walls with metal ties or by providing a masonry bond. Where masonry bond is used, it is necessary to build the partitions at the same time the exterior walls are laid up or to provide openings in every other course to bond the partition. There are three general disadvantages of masonry bond for tying partitions to exterior walls as follows:

First, the partitions are usually not all of the same thickness—for instance, load-bearing partitions may be 8 in. thick while nonload-bearing partitions are generally 4 in. thick. This necessitates cutting the exterior wall units which increases construction costs.

Second, cracking may result from settlement of the floor supporting the partition as the walls are rigidly fixed at their juncture.

Third, as the floors supporting non-load-bearing partitions frequently do not start at course height, it is necessary to start the partition with fractional height units which involves cutting standard height units.

For these reasons, metal ties are recommended for bonding interior partitions to exterior walls. These ties are flexible enough to allow for slight relative movements of either wall.

EXPANSION AND CONTRACTION JOINTS

Expansion joints are rarely needed in concrete masonry walls. Concrete masonry units are seldom placed in a wall in a fully contracted condition and usually some shrinkage takes place after the walls are built. Volume changes in concrete masonry walls are due to two conditions: temperature variations and changes in moisture content in the individual masonry units and in the mortar. A wall 100 ft. long

built of green concrete masonry units may shrink as much as .8 in. as it dries out due to moisture changes alone. With an average thermal coefficient of expansion of .0000040 and an increase in temperature of 100 F., this same wall will expand only about .48 in. It can be seen, therefore, that expansion is seldom serious in concrete masonry walls but that contraction may cause damage.

It is advisable to provide contraction joints for long walls, possibly for walls more than 50 ft. long. In order to be most effective, provision for contraction should be made throughout the entire cross-section of the building, including foundation walls, floors and roof. The location of the contraction joints will depend on the building plan. Some architectural treatment, such as providing vertical drains in front of the joints can often be used to hide them.

There are several types of contraction joints available. One type of joint can easily be made by laying up the wall in the usual manner except that half and full length units are used to provide a continuous vertical joint. The ends of the masonry units on one side of the joints are painted with a tar of asphaltic material to prevent mortar bond. The outside of this joint is raked to a depth of about ½ in. and filled with a mastic calking compound fof waterproofing the joint.

PARAPETS AND FLASHINGS

Most flat roofed buildings have parapet walls which may be a contributing factor to cracking if not properly designed and built. A parapet wall is in an exposed position, much more exposed to heat and cold than the walls below. During hot weather both sides of the parapet are exposed to the sun and therefore has a much higher temperature than the wall below which is exposed to the sun from only one direction and has also the moderating influence of the cooler air inside the building.

On hot days the parapet wall expands some amount depending on its temperature and the coefficient of thermal expansion of the wall material. At night when the temperature falls the parapet wall contracts more than the walls below. However, the parapet wall will probably not come back the full amount it expanded due to the weakness of masonry walls in tension and the friction between the parapet and the roof. Small cracks then develop. As this process is continued the parapet wall will creep out further as the cracks fill with dirt and results in an overhang and a wider opening of cracks already formed or the opening of additional cracks. This same type of reaction to daily or seasonal changes in weather is true of coping units. The appearance of the cracks and the overhang of the parapet is unsightly and the cracks may permit water to enter the wall.

The problem is one of relative movement of coping and parapets with respect to their supports. This may be corrected by the proper use of expansion joints and by some means of anchoring the coping and parapet to their supports. Various means of accomplishing these results can be used.

Where there is a concrete roof, small piers or pedestals may be built integrally with the slab at the corners. By building the masonry parapet walls against and around these piers, movement due to expansion of the parapets is limited. The same effect may be obtained by building serrations around the perimeter of the roof slab. Another method of accomplishing this same result would be to place dowels in the concrete roof slab to project into the cores of the concrete masonry units. Cores in which these dowels occur are filled with mortar or concrete, thus effectively tying the parapet to the roof slab and controlling movement. When a flat wood roof is used, this result can be accomplished by providing reinforcement in the cores of the hollow concrete masonry units, extending the bars through the parapet and a sufficient distance into the wall below to provide a tie.

Some engineers feel that it is worth while to provide horizontal reinforcement in parapet walls. The provision of ¼-in. round rods in each face shell mortar joint in each course of the parapet would probably be worth while from the standpoint that they would help to prevent the wide stepdown cracks frequently seen near the corner of these walls. This reinforcement should only be needed for a distance of approximately 20 ft. from the corners.

Coping units can be provided with a slot to receive a dowel from the parapet. Expansion joints should also be provided between the coping units in order to localize the movement and to reduce the resulting stresses. The average coping unit is approximately 4 feet long and a ½-in. expansion joint for every third unit should be sufficient.

FLASHING

Flashing should be provided wherever there is danger of leakage and should consist of copper, rust-resisting metal or metal painted with a rust-inhititive paint. Points requiring protection by flashing are: tops and sides of projecting trim; under all vertical joints in sills, coping and caps at intersection of walls and roof; over windows; and under horizontal surfaces which may permit the accumulation of water on or the passage of water through them.

Through flashing should always be used under the coping. This flashing should extend through to the outside and be bent down to form a drip. On the inside of the parapet the flashing should be bent down over the roofing to form counterflashing. Flashing should not

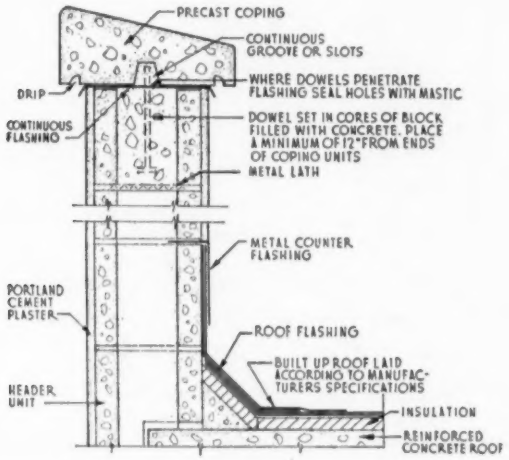
PRECAST COPING
CONTINUOUS GROOVE OR SLOTS
WHERE DOWELS PENETRATE FLASHING SEAL HOLES WITH MASTIC
DRIP
CONTINUOUS FLASHING
DOWEL SET IN CORES OF BLOCK FILLED WITH CONCRETE. PLACE A MINIMUM OF 12" FROM ENDS OF COPING UNITS
METAL LATH
METAL COUNTER FLASHING
PORTLAND CEMENT PLASTER
ROOF FLASHING
BUILT UP ROOF LAID ACCORDING TO MANUFACTURERS SPECIFICATIONS
HEADER UNIT
INSULATION
REINFORCED CONCRETE ROOF

Fig. 2—Typical coping detail

be placed between the parapet wall and the roof slab as this provides a plane of weakness which will not be able to resist the creeping action of the parapet.

Flashing over the windows is important where parapet walls are used because this is one of the first points where leakage will appear. Water which has come through the coping, the parapet or the wall itself flows down the cores until it strikes the lintel and is deflected into the building. The detail of flashing at this point will depend on the type of lintel used, whether one piece or split. The flashing need not extend more than 12 in. beyond either side of the opening and will usually be stopped an inch from the outside face of the wall.

DRAINAGE

If the building is in poorly drained soil, it is advisable to provide a line of drain tile around the footings having a slope of ½ inch in 12 feet and connected to a suitable outlet. It is generally desirable to parge the earth side of concrete masonry basement walls to increase watertightness. This parging should consist of two ¼ in. thick coats of portland cement plaster using the proportions of 1 part portland cement to 2½ parts mortar sand. At the juncture of the outside wall face and footing, the cement plaster should be thickened and rounded to provide a cove to divert water from the base of the wall. Backfilling should not be undertaken until the first floor is in place and the plaster parging has hardened sufficiently to prevent damage.

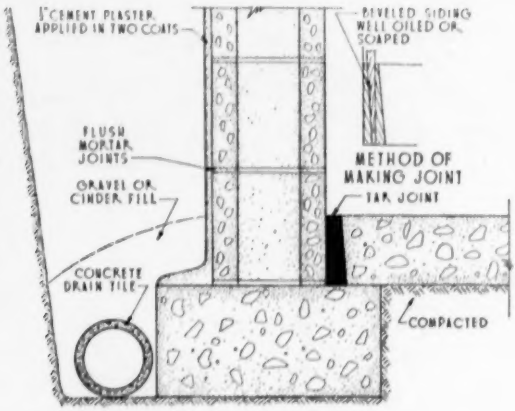

Fig. 3—Exterior basement wall construction for wet soil conditions

MORTAR JOINTS

Treatment of mortar joints in exterior concrete masonry walls is important in building a watertight wall. Joints may be tooled or untooled. In the latter class is the struck joint which is made by cutting off the excess mortar with a trowel, and the raked joint which is a recessed joint made by raking out the mortar to a depth of ¼ in. or more. Struck or raked joints greatly increase the chances for leaks. In making these joints there is a tendency to open up the body of the mortar and to draw it away from the masonry unit forming small ledges upon which water can collect. Struck joints are also likely to be torn and drawn away from the units. If these joints are used, adequate means of waterproofing or parging should be provided, or special attention given to the selection of material and tooling of joints.

A tooled joint is one in which the mortar has been smoothed or worked by stroking the mortar with a mortar tool or trowel. There are four common types of tooled joints, namely the "V", concave, square, and weathered joint. The principal advantage of tooling the mortar joints is that the mortar is forced against the masonry unit which increases the bond. The usual procedure is to strike the joint as the units are laid up and to delay the tooling until the mortar has perceptably hardened. This allows the mortar to take most of its shrinkage before it is forced against the units. Another advantage of tooling is that the mortar is compacted and smoothed into a dense surface texture which will more readily shed water. Tooling also will reveal weak points in the mortar joints as the pressure used in tooling will often push the mortar entirely out at points where there is only a thin shell of mortar.

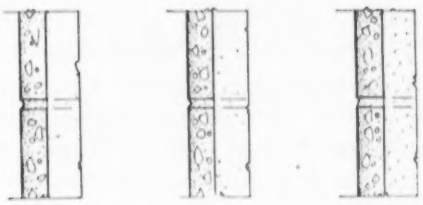

Fig. 4—Approved tooled mortar joints. A horizontal effect is produced by tooling horizontal and striking vertical joints

Thin mortar joints are best, because such joints produce a stronger, more watertight wall.

No mortar, however good, can make up for defective design and poor workmanship. Leaky walls are not more common with one type of mortar than another. While some mortars are better adapted than others to particular jobs, the differences in the watertight properties are minor compared with design and workmanship.

Tight joints are essential to watertight masonry. Bed joints must be full and level and not furrowed. Head joints should be carefully buttered to fill the joints tightly. If all the joints in the exposed face of the wall are tightly filled, there is little possibility of leakage unless the design is faulty.

PORTLAND CEMENT PAINT

The most popular method of waterproofing concrete masonry walls is by the application of two coats of portland cement base paint. This paint also provides a decorative function and is available in a wide range of colors. Other types of paint have also been used for painting concrete masonry, but it has been found that portland cement base paints are generally more satisfactory and economical. The use of a portland cement base paint meeting the Federal Specifications for Paint; Cement-Water, Powder, White and Tints, which was issued June 30, 1941, is recommended. The manufacturer of the paint should specify that his product meets the Federal Specifications.

The most important considerations for obtaining a satisfactory paint job is the proper application and curing of the paint. Where properly applied, these paints bond to and become a part of the concrete masonry and will give satisfactory water-proofing performance for many years.

It would require too much space to discuss properly the correct methods of applying and curing portland cement base paints. The Portland Cement Association has prepared an information sheet, "Sug-

gested Specifications for Application of Portland Cement Paint on Exposed Concrete Masonry Walls", which discusses this subject.*

Concrete masonry buildings can be built waterproof and crackproof by combining high quality with proper design and good construction. A logical explanation for the few failures which have occurred is the haste with which our modern buildings are erected. Workmanship is frequently sacrificed for speed. Perfection in design and materials cannot make up for this sacrifice.

The construction of good walls must begin with the design of the building, follow through the selection of materials and the supervision of the workmanship, and continue with the maintenance of the structure after its completion.

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*ACI Committee 616, Recommended Practice for the Application of Paint to Concrete Surfaces" has a report in preparation.

Discussion of this paper should reach the ACI Secretary in triplicate by June 1, 1942, for publication in the JOURNAL Supplement, concluding Proceedings V. 38.

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN (Supplement) November, 1942

Discussion of a paper by Kenneth C. Tippy:

Good Practice in Concrete Masonry Wall Construction*

DISCUSSION (at the Convention) by the AUTHOR with supplementary information, F. O. ANDEREGG, BENJAMIN WILK, J. C. PEARSON, DOUGLAS E. PARSONS, (after Convention) AUSTIN CRABBS and AUTHOR'S CLOSURE.

By KENNETH C. TIPPY†

This paper discussed some of the construction details necessary for good results in concrete masonry construction. I would like at this time to add some material supplementary to that presented.

The main problem in concrete masonry construction is to build strong, durable, watertight walls. That concrete is an excellent building material is evidenced by the many satisfactory buildings which have been built with concrete units as the sole or principal wall material. In the main, these buildings have not cracked or leaked, or such failures as have occurred have been of a limited nature. Such failures as we have investigated have indicated substandard masonry units, poor workmanship or faulty design. No building material available today is proof against these faults.

MOISTURE CONTENT AND SHRINKAGE

All manufactured masonry materials shrink during the process of manufacture: clay products while in the drying room and in the kiln; concrete products during the curing and drying out period. This shrinkage is a definite part of the manufacturing process and neither material should be considered a finished product until a large part of this shrinkage has taken place. Concrete masonry units laid in a wall in a green or wet state will later dry out and the resulting shrinkage is frequently sufficient to cause cracking.

Studies made in the Portland Cement Association laboratories and reported through American Concrete Institute‡ showed that the moisture content of concrete masonry units at the time they are laid in the wall has an important bearing on the extent of shrinkage. These studies

*ACI JOURNAL, Feb. 1942; *Proceedings* V. 38, p. 317.

†Formerly of the Portland Cement Association, Chicago; now Assistant Professor, Dept. of Civil Engineering, Syracuse University, Syracuse, N. Y.

‡Shrinkage Measurements of Concrete Masonry, W. D. M. Allan, ACI JOURNAL, Apr. 1930; *Proceedings* V. 26, p. 699.

included tests of 24 panels of concrete masonry units made with three different aggregates. The following example is typical of the panels tested:

Panels erected with saturated two-day old units containing 19.3 per cent moisture by weight contracted .54 in. per 100 ft. in 45 days. Further shrinkgae due to the heating of the building in which the panels were housed resulted in a total shrinkage at 150 days of about .67 in. per 100 ft.

Panels erected with seven-day old units containing 7.3 per cent moisture contracted about .35 in. per 100 ft. in 30 days. Total contraction after the building was heated was about .50 in. at 150 days.

Twenty-eight-day old units when erected in panels contained 4.9 per cent moisture. Total shrinkage for 150 days was .35 in. per 100 ft.

Panels erected with 90-day old units containing 2.8 per cent moisture expanded slightly during the first two days before contraction started. Total shrinkage was about .17 in. per 100 ft.

These studies showed that air drying of the units for five days reduced the shrinkage for the 150-day period to about 75 per cent of that obtained with saturated units. Similarly, 26 days of air drying reduced the shrinkage to about half that obtained with saturated units. Eighty-eight days of air drying reduced the shrinkage to about 26 per cent of that of the saturated units.

From these data it is evident that concrete masonry units should be in as near an air-dry condition as possible prior to being used. The use of units only partially dry, containing 2 to 5 per cent of moisture by weight, will result in the major amount of shrinkage which will occur in a concrete masonry wall. Present Federal and A.S.T.M. Specifications limit the moisture content at the time of delivery to the job to 40 per cent of the total possible absorption and this figure is generally accepted by others. This is approximately twice as much moisture as is contained in fully air-dry units. Unfortunately, however, too many products manufacturers send out green, uncured or wet units and trouble frequently results. Tests on forced air drying of concrete masonry units conducted by the Portland Cement Association have shown that these units are in a fully air-dry condition when they contain from 15 to 25 per cent of their total absorption.

Some of the points I plan to discuss are best shown by pictures and drawings.

Fig. A shows some of the results of tests on forced air drying of concrete masonry units using three different stockpiles of 8 x 8 x 16-in. blocks. Pile No. 1 with the cores vertical was placed directly on the ground, a practice common to a good many products plants. Piles No. 2 and 3 were set on 8-in. high piers so that air could enter the cores at the bottom and travel vertically through the pile by chimney action. You will note that the moisture content of the units in Pile No. 1 was

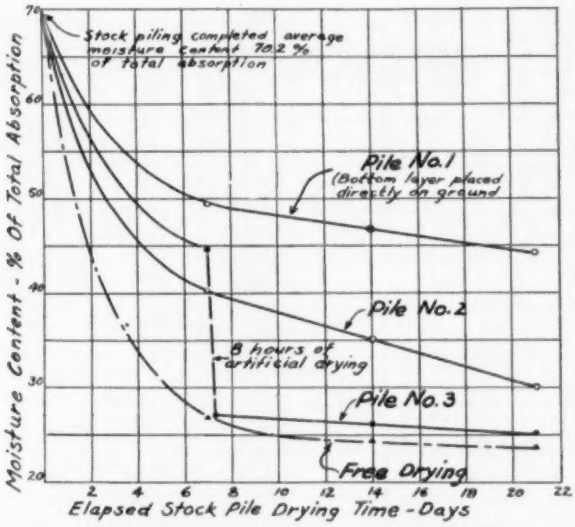

Fig. A—Comparison of results for piles No. 1, 2 & 3

8 x 8 x 16-inch-3 oval core units of Pottsco concrete used. Piles contained 960 units, approximately 8 ft. wide by 11 ft. long by 6 ft. 8 in. high; units stacked with cores vertical and aligned; artificial drying applied with bottom air injector type dryer. Free drying pertains to selected units placed apart from pile for maximum air drying.

about 44½ per cent of the total possible absorption at the end of 21 days. Pile No. 2 contained 40 per cent moisture at seven days and 30 per cent at 21 days. Forced air drying was applied to Pile No. 3 for eight hours after it has been in the stockpile for one week and the moisture content was reduced during this eight-hour period from 45 per cent to 27 per cent.

Fig. B shows the comparison of results for Piles No. 4 and 5 which consisted of 8 x 8 x 16-in. units with their cores horizontal. My only purpose in showing this figure is to indicate that there is a definite lower limit for moisture content in concrete masonry units which will, of course, depend upon temperature and humidity conditions. It will be noted that Pile No. 4 was froced air dryed to a moisture content of approximately 22 per cent at the end of seven days and that it then absorbed moisture from the air until it contained about 23 per cent of moisture.

WATER TIGHT WALLS

The subject of leaky masonry walls is a matter with which most of us are interested. Probably no other problem in the construction industry causes so much annoyance. This is because the prevalence of leaks is not usually great enough for builders to take positive action to prevent them.

Tests made by the Development Department of the Portland Cement Association on the resistance to rain penetration of more than 200 masonry

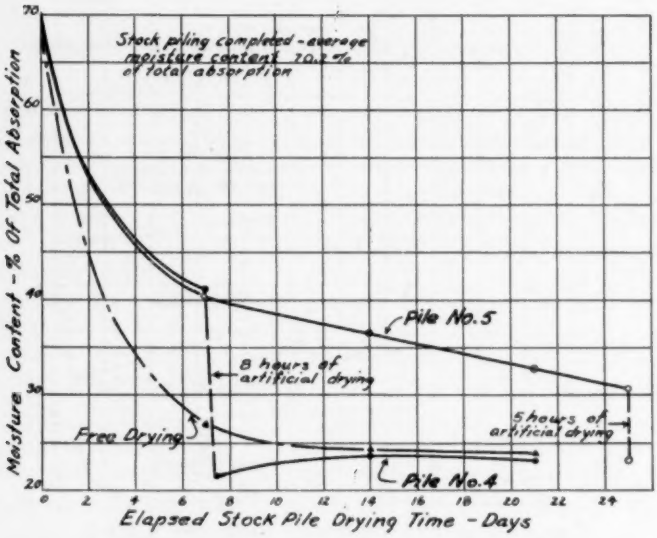
Stock piling completed - average moisture content 70.2 % of total absorption
Moisture Content - % Of Total Absorption
Pile No. 5
8 hours of artificial drying
5 hours of artificial drying
Free Drying
Pile No. 4
Elapsed Stock Pile Drying Time - Days

Fig. B—Comparison of results for piles No. 4 and 5

8 x 8 x 16 in.—3 oval core units of Pottsco concrete used. Piles contained 1000 units and measured 6 ft. 8 in. wide by 14 ft. long by 6 ft. 8 in. high. Units stacked with cores horizontal and aligned from front of pile to rear. Artificial drying applied with end air injector type dryer. "Free drying" pertains to selected units placed apart from pile for maximum air drying.

wall panels indicated that the class of workmanship used in laying up the walls was the most important factor influencing the performance of brick walls. Two grades of workmanship were used—average and excellent. Average workmanship was characterized by furrowed bed joints, buttered head joints, incomplete filling of the joints between wythes and rather less careful tooling of the joints. Excellent workmanship provided full and carefully leveled bed joints, full and shoved head joints, solid filling of the longitudinal joint between wythes and careful tooling of the joints after the mortar had started to stiffen perceptibly.

I would like to illustrate these types of workmanship by Fig. C, D, E, F. and G. You will note (Fig. C) that the concrete masonry backup is being laid with a furrowed mortar joint and that the space between the face brick and the backup is left open. Should moisture be able to penetrate this brick facing, and it occasionally is, it can impound against the backup material and pass through the wall. Fig. D is a rear view of the face brick laid with average workmanship and properly illustrates the openings through which moisture can penetrate.

Fig. E illustrates the same type of wall built with excellent workmanship. Note the spread horizontal joints which provides full bearing for the brick and that the space between the brick and backup wall is com-

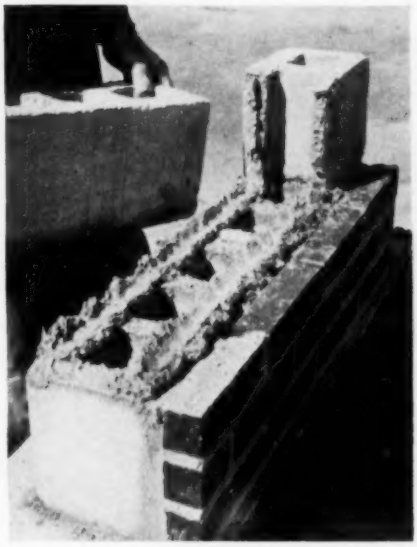

Fig. C

pletely filled with mortar. Head joints are also completely full and it would be very difficult for moisture to penetrate a wall of this type.

Fig. F shows a wall with 4-in. face brick and 8-in. of concrete masonry backup showing the use of header block. Note that the joints between the brick header units are completely filled making it very difficult for water to pass between the brick units.

Fig. G providing a side view of a completed wall, is a very good cross section of one built with excellent workmanship. Note that the wall consists of 4-in. of face brick with all joints full, ½-in. of plaster and 8-in. of concrete masonry with full headers in every seventh brick course. This wall gave excellent results in the rain penetration studies.

Treatment of mortar joints in exterior concrete masonry walls is important in building a watertight wall. (Fig. 4 of original paper) Joints may be tooled or untooled. In the latter class is the struck joint which is made by cutting off the excess mortar with a trowel and the raked joint which is a recess joint made by raking out the mortar to a depth of ¼ in. or more. Struck or raked joints greatly increase the chance for leaks. In making these joints there is a tendency to open up the body of the mortar and to draw it away from the masonry unit forming small ledges upon which water can collect. Struck joints are also likely to be torn and drawn away from the units.

In tooled joints the mortar has been smoothed or worked by stroking with a mortar tool or trowel. There are four common types of tooled joints, namely, the "V," concave, square, and weathered joint. The principal

Fig. D (top left)
Fig. E (bottom left)
Fig. F (above)

Fig. G

advantage of tooling the mortar joints is that the mortar is forced against the masonry unit which increases the bond. Another advantage of tooling is that the mortar is compacted and smoothed into a dense surface texture which will more readily shed water. Tooling will also reveal weak points in the mortar joints, as the pressure used in tooling will often push the mortar entirely out at points where there is only a thin shell of mortar. This slide shows three of the more common types of tooled joints.

FLAT ROOFS, PARAPET WALLS AND FLASHING

Most flat roof buildings have parapet walls which are often a contributing factor to leaking and cracking if not properly designed and built. A parapel wall is in an exposed position, much more exposed to the extremes of the weather than the wall below. During the hot seasons the parapet wall expands somewhat, depending upon its length, the temperature and the coefficient of thermal expansion of the wall material. When cooler weather prevails the wall contracts, but will probably not come back the full amount it expanded. This is because of the friction of the wall on its support and the weakness of the masonry in tension.

Fig. H is of a residence in Indiana which was inspected in 1940. The complaint was of leakage occurring in the plaster at the heads of corner windows in three bedrooms under the roof deck. The inspection showed that the coping units had separated from their supports for almost their entire length and that cracking had occurred in the head joints. This of course, permitted water to pass through the coping and travel down the vertical cores of the units until it reached the window heads, staining

Fig. H

the plaster. Some water also probably entered the wall cracks shown in the illustration. At points other than where windows occurred, the moisture traveled down the cores until absorbed by the units.

The lower crack is at the roof line and was caused by the expansion of the low parapet wall. Typical step-down cracks can be observed at the corners of the building.

There are several means of correcting these difficulties. When there is a concrete roof, small piers or pedestals may be built integrally with the slabs at the corners. By building the masonry parapet walls against and around these piers movement due to the expansion of the parapet is limited. The same effect may be obtained by building serrations in the slab near the corners. Another method of accomplishing this same result is to place dowels in the concrete roof slab to project into the cores of the concrete masonry units. Cores in which these dowels occur are filled with concrete, thus effectively tying the parapet to the roof slab and controlling movement. Where a flat wood roof is used, this result can be accomplished by placing reinforcement in the cores of the hollow concrete masonry units extending the bars through the parapet and a sufficient distance into the wall below to provide a tie. This reinforcement should be necessary only for a distance of three or four feet from the corners. Expansion joints should also be provided between the coping units in order to localize movement.

Fig. I shows a preferred flat roof construction as it eliminates the parapet wall and the difficulties usually arising from its use. The insulation placed on top of the concrete slab protects it from shrinkage and expansion due to weather changes. Flashing into the gutter prevents leakage.

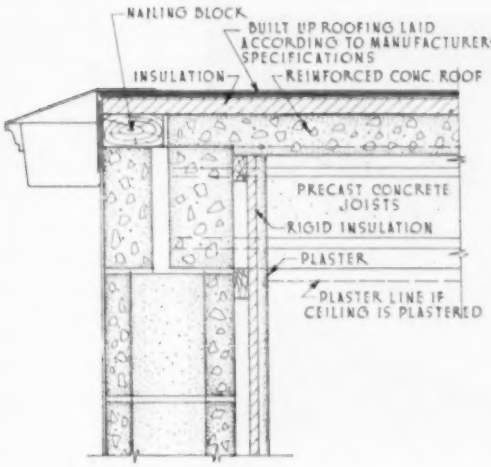

Fig. I

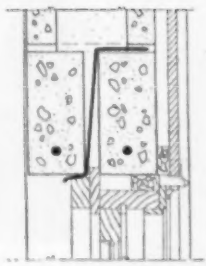

Fig. J

Flashing over the windows is important because this is one of the first points where leakage will appear. Water which has come through the coping, the parapet or the wall itself will flow down the cores until it strikes the lintel and is deflected into the building. The detail of flashing at this point will depend on the type of lintel used—whether one piece or split. Fig. J shows a suggested flashing detail for a split lintel.

WALL REINFORCEMENT

Some concrete products manufacturers have advocated the use of reinforcement in some of the horizontal mortar joints of concrete masonry wall as a means of preventing cracking. (Fig. 1, original paper) Some of these manufacturers have used such reinforcement for a period of several years and claim it has been effective in preventing cracks.

There are no data available, however, to prove this contention and such statements are based on the experience of individuals with certain jobs. Some observers think it is of no particular value; others seem to think that this type of reinforcement distributes stresses and controls cracking to the extent that cracks are finer and more widely distributed.

Quarter-inch round pencil rods placed in each face shell mortar joint of the course so reinforced are generally used and in the past year a commercial type of reinforcement consisting of two longitudinal wires welded to diagonal cross wires has been developed. (Fig. 1 of original paper) In so far as such reinforcement increases the tensile strength of the wall, it should help to prevent the formation of wide cracks at critical sections. It should never be regarded as a substitute for proper curing and drying of the units or as a "cure-all" for poor design and construction. In the light of our present knowledge such reinforcement should

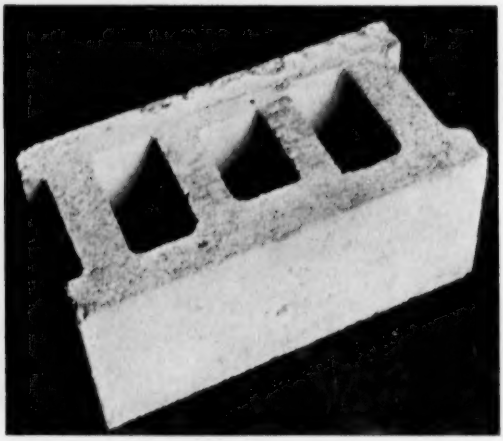

Fig. K

not be regarded as anything more than a supplementary aid in controlling cracks.

It is generally observed that vertical cracks in concrete masonry walls start at a vertical joint between two units cracking the concrete masonry units in alternate courses through the face shell at the center core. The use of thin face shells has aggravated this condition. There has been some discussion about thickening face shells and some products manufacturers have even considered going back to the two-core unit for this reason. Fig. K is from a photograph of a concrete masonry unit made in Davenport, Iowa. The face shell at the center core is considerably thicker than at the ends. The purpose of this is to strengthen the unit at the center so that it will be able to resist cracking at this point. The theory is that cracks will travel around such units following the mortar joints. We are watching this development with considerable interest to see if it proves effective.

Concrete masonry buildings can be built waterproof and crackproof by combining high quality units with proper design and good construction. A logical explanation for the few failures which have occurred is the haste with which our modern buildings are erected. Workmanship is frequently sacrificed for speed. Perfection in design and material cannot make up for this sacrifice.

The construction of good walls must begin with the design of the building, follow through the selection of materials and the supervision of the workmanship and continue with the maintenance of the structure after its completion.

By F. O. ANDEREGG†

This paper is an excellent review of the subject and most of Mr. Tippy's recommendations are correct. I would however, like to raise some points in regard to the choice of mortar for setting concrete blocks. Judging by the discussion of the last 10 years, most people seem to be interested in using only very strong mortars; but other mortar properties should receive careful consideration. First, I should like to ask, "Which should be stronger, the units or the mortar?" You may be surprised to learn that a 1:1:6 mortar contains more cement per volume than most concrete units. Such mortars readily attain strengths of 5,000 psi. in the joint. How many concrete blocks can match this value? The modulus of elasticity of such joint mortars, according to our measurements, runs about 1,000 times their compressive strength. Is it fair to the blocks to embed them in mortar of such high rigidity, or should more cushioning action be provided to help take up part of the movements, which masonry walls are constantly undergoing? The load bearing capacity of even 1:2:9 mortars, in the thin joints common with concrete units, is ample. The important question involves durability. This is greatly helped by the thinness of the joint and by removal of excess moisture from the mortar by the units. More attention should be focused on balancing mortar rigidity against mortar durability.

Mr. Tippy discusses wall leakage. Most of the leakage occurs between the units and the mortar because of incomplete contact. If the mortar is too lean, or too dry, or too non-plastic, or if the units have too great a suction power, contact is apt to be poor. I noticed in the slides shown by Mr. Tippy that mortars of excellent plasticity had been chosen. Judging by their appearance they would run about 1:2:7½ or 1:1:4½. A block having a high initial suction tends to congeal the mortar surface at the first instant of contact, especially in hot weather. The congealed mortar makes only point contact with the blocks so that both bond strength and watertightness are adversely affected. Such blocks should be wetted, but this is often difficult to attend to on the job. This fact should be considered when drawing up specifications for the blocks.

The author is correct in recommending the elimination of parapet walls. They stick up like sore thumbs in more ways than one. If it is necessary to erect one, it should be built hollow with through-flashing above the snow line and with weepholes out on to the roof. This will prevent the accumulation of that moisture which seems to be so prevalent in and to cause the disintegration of, most prapet walls.

†Consulting Specialist on Building Materials, Newark, Ohio.

By BENJAMIN WILK*

There is the question of shrinkage. From a commercial standpoint it is practically impossible in a large plant to store block more than 30 days, and it would, therefore, be desirable to study possibilities of less shrinkage through other means than long time storage. Perhaps we have been emphasizing saturated block in the curing rooms too much. A closer regulation of the moisture content when the blocks are taken from the curing room might be helpful. We have been using natural gas for curing during the past two years and find we can control the amount of moisture in the block a great deal better than with steam. Something along this line may be the solution rather than long time yard curing.

By J. C. PEARSON†

It seems to me that the prediction and control of movements in masonry walls, and particularly in masonry walls built of concrete units, will have to depend on more quantitative knowledge of such movements than is now available. This knowledge can hardly be obtained in the laboratory alone; true, we may learn something of the relation between shrinkage and moisture content of units, and something of thermal coefficients, but the effect of restraint in the built up masonry has almost necessarily to be determined from field measurements. Thus the answer to a question asked in the Quiz Session, viz, how much shrinkage would be permissible in concrete units to give reasonable expectation that long walls will not crack, is not known—we need some long walls, built with tested materials under recorded conditions, and fitted with gage points for strain measurements. One can only guess at the answer now, but a lot could be learned from such experiments. The effect of light reinforcement in horizontal joints, now controversial, is another question that could be pretty well settled by field scale measurements. Unfortunately, the opportunity for such tests is rare in commercial jobs, and test structures are expensive. Still the need is great, and we shall do well to watch for any opportunities that present themselves.

By DOUGLAS E. PARSONS
(presiding officer)

One of the graphs indicated that the moisture content of the units after prolonged drying was about 25 per cent. Does not the rate of drying and the final moisture content depend importantly on the type of aggregate?

MR. TIPPY

Yes it does. The rate of drying of the unit will depend not only upon the type of aggregate but also upon its coarseness or fineness. Wind

*General Manager, Standard Building Products Co., Detroit.
†Director of Research, Lehigh Portland Cement Co., Allentown, Pa.

conditions, temperature and relative humidity are also important. A hot, dry wind would do much to speed the drying of the units. Incidentally the blocks referred to in my discussion of forced air drying were made of an expanded slag aggregate.

MR. PARSONS

Photographs illustrating walls with excellent workmanship showed that the bed joints were full and that the space between the facing and the back-up units was filled with mortar. Tests conducted at the National Bureau of Standards indicated that the filling of the space between the facing and the back-up units provided an effective barrier against moisture penetration. However, it was very difficult to fill this space, and with certain types of units it was almost impossible. Masons, engineers and inspectors all agree that it is impractical to fill the space without the use of an outside scaffolding, which often would add considerably to the cost of construction. Fortunately, the results of the tests indicated that the complete filling of the space was not necessary to secure a satisfactory barrier against moisture penetration. The application of a thin but complete parging of mortar, either to the back of the facing or to the face of the backing, was sufficient and was obtained without difficulty. Moreover, direct comparisons between the performances of walls, some of which had the bed joints for the brick work furrowed and others spread smooth and level indicated that the furrowing of the bed joints did not increase permeability significantly. The essential attributes for good workmanship are: the complete filling of the head joints in the brick work, the application of mortar to both the inner and the outer edges of vertical joints in hollow back-up units, and the wetting of absorptive brick before they are laid. For best results, the mortar should be as wet (high a flow or slump) as the mason can use it conveniently. That is, the mortar should be retempered as often as is needed to maintain the proper consistency, provided set has not begun.

By AUSTIN CRABBS*

This paper by Mr. Tippy is something that everyone interested in good concrete masonry walls should read and re-read. However two points, very important to a discussion of this kind, have not been covered. The first point is in reference to the specification for 40 per cent contained moisture and the second is the relation of quality of mortar to the absorptive qualities of the concrete masonry units being used with a given mortar.

As to the 40 per cent contained moisture specification, this is a step in the right direction, but research to date, on which this specification is based, is far from complete. It is this writer's contention that this specifi-

*Austin Crabbs Inc., Davenport, Ia.;—by letter, subsequent to convention.

cation definitely and unjustly penalizes the manufacturer of a particularly high grade block. For example, a 45 lb. block with 10 per cent absorption will take up 4½ lb. of water and 40 per cent of this amount is 1.6 pounds. A block of the same weight with 5 per cent absorption will take a total of 2.25 pounds of water and 40 per cent of this is 0.9 pounds. The writer so far has not seen a 5 per cent absorption block that would dry out to this extent in normal air drying. But, on the other hand, the higher the absorption, the quicker and easier it is to reach a figure of 40 per cent contained moisture.

If the 5 per cent moisture block were obtained by additional quantities of cement, then this 40 per cent specification might in all cases be justified, as shrinkage varies with richness of mix. On the contrary, however, most manufacturers of blocks with 5 per cent absorption or under, achieve this result through careful grading of aggregate and usually employ leaner mixes than the manufacturers of a 10 per cent absorption block, who as a rule use but one aggregate often of very poor grading.

As to the second point regarding quality of mortar, this too should be considered in connection with the absorptive characteristics of the units used with a given mortar. To illustrate, a fairly poor mortar may bond tightly to a moderately absorptive unit, whereas a very high grade mortar may make a poorer bond if the units are extremely dense and smooth. Thus quality of mortar and quality of the units should be considered together in deciding on mortar for a particular job.

The two points discussed above may be summarized as follows:

1. Further investigation should be carried on to determine the shrinkage behavior in drying on concrete masonry units of lean mixes and very low total absorption. Then the 40 per cent contained moisture specification should be revised on the basis of facts secured from such an investigation.

2. In view of the fact that there has been some consideration of the use of poorer mortars to prevent wall cracking from shrinkage of units in the wall, I suggest that the same result can be obtained by using units of lower absorption and smoother joint surfaces, in which case a better quality and hence more durable mortar would be used. We have made considerable progress toward the rules required to secure good concrete masonry walls, but much research work along lines indicated above, is still needed.

MR. TIPPY

I thank those who discussed this paper for their contributions to the highly controversial problem of what constitutes good practice in concrete masonry construction.

In answer to Mr. Anderegg: I intentionally steered clear of recommendations for mortar proportions. Thousands of tests have been made on various masonry mortars and innumerable reports are available for those who care to read them. The author is inclined to agree with reports of the U. S. Bureau of Standards and with results obtained from tests conducted by the Portland Cement Association to the effect that good results from masonry mortars depend more upon good workmanship than any other factor. All of the walls shown in the slides referrred to by Mr. Anderegg were laid up with either a 1:1:6 or 1:0.25:3 mortar.

Mr. Wilk believes it is practically impossible for a products plant to store blocks more than 30 days and I agree with him. The trouble experienced with shrinkage of concrete masonry units has not been due to the fact that they are only 30 days old but to the practice of some manufacturers of sending out block which are less than 7 days old. On the other hand the fact that block are more than 28 days old does not insure that they will not shrink as it is quite possible with open storage and a rainy period for the block to be almost completely saturated at that age. The only satisfactory solution for this problem is either covered storage and block approximately 28 days old, forced air drying or high-pressure steam curing.

The author heartily agrees with Mr. Pearson regarding the need for cracking studies of full scale walls. Products manufacturers would do the industry a great service if they would note the performance of large structures built with their units, and make the information obtained available to the industry.

With regard to Mr. Crabbs reference to the 40 per cent contained moisture requirement: this matter has been discussed. There is a growing sentiment that this 40 per cent limitation is too high and that it should be lowered to something in the neighborhood of 25 or 30 per cent. The author is inclined to agree with this lowered moisture figure as it would undoubtedly require something considerably better than the open yard curing so prevalent today. On the other hand Mr. Crabbs is probably correct in his contention that the permissible moisture content in terms of total absorption should be lower for heavyweight aggregate block than for the lightweight block.

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The Effect of Belt Transportation on Concrete Aggregate Grading*

By GORDON L. WILLIAMS†

SYNOPSIS

Steps in the solution of the problem presented by breakage of graded concrete aggregate through 26 "flights" of a 9½-mile belt conveyor which took aggregate from the gravel plant at Redding, Calif., to the mixing plant for Shasta Dam.

The Redding-Coram conveyor, which transports graded concrete aggregate 9.6 of the 10.8 miles between the gravel plant at Redding and the mixing plant at Shasta Dam,‡ consists of a series of 26 individual belts or "flights". Each transfer between flights contains a drop of about 10 feet. Including the two end transfers, this makes a total drop of some 270 feet, through which the aggregate falls. The potential energy—about 300 horsepower—must be dissipated in a manner which will not break the gravel, wear the chutes, or damage the receiving belt to any great extent.

It is believed that the magnitude of this problem was not generally appreciated when the conveyor began operating on May 6, 1940. Unfortunately, the entire conveyor line, including all of the transfer stations, was completed before any portion of it was tested under load. The original chute idea (at I in Fig. 1) appeared plausible at the time. It consisted of two chutes, the lower one acting as a drip chute to take the load while the discharging belt was starting; when the belt reached full speed, the aggregate was thrown onto the higher chute. The intent was to have the chute at such a slope that the sliding friction would approximate the gravitational accelerating force. In that way the gravel would flow down the chute at a uniform rate of speed, its horizontal component being about equal to the velocity of the receiving belt—550 ft. per minute.

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†Formerly Associate Engineer, Bureau of Reclamation; now with Tennessee Valley Authority.
‡Shasta Dam is being constructed by the Bureau of Reclamation on the Sacramento River in Northern California as a part of the Central Valley Project.

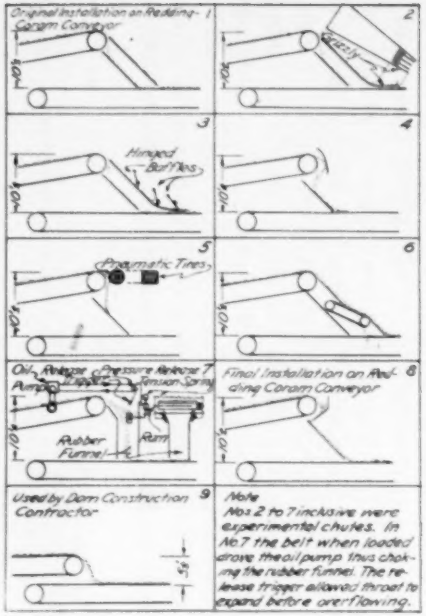
Original Installation on Reading Coram Conveyor
Gravity
Hinged Baffles
Pneumatic Tires
Oil Pump
Pressure Release Tension Spring
Rubber Funnel
Ram
Final Installation on Reading Coram Conveyor
Used by Dam Construction Contractor
Note
Nos. 2 to 7 inclusive were experimental chutes. In No. 7 the belt when loaded drove the oil pump thus choking the rubber funnel. The release trigger allowed throat to expand before oren flowing.

Fig. 1—Conveyor transfer chutes—steps in the problem of reducing aggregate breakage

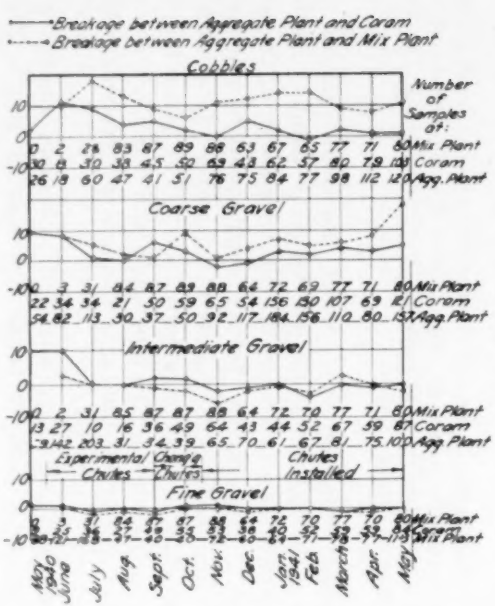

Fig. 2—Diagram to show breakage of graded gravel in transit

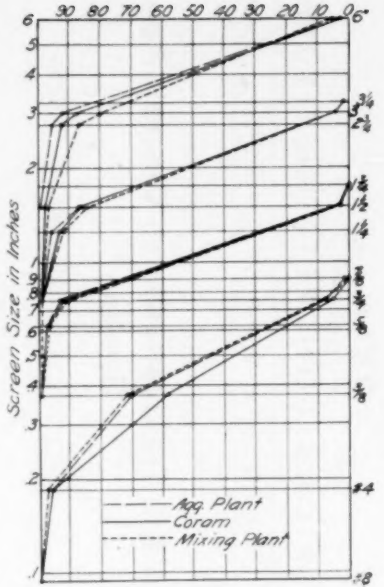

Fig. 3—Grading curves on gravel at Reading gravel plant, at Coram hoppers and at the mixing plant

As is seen in Fig. 2, the breakage was about 10 or 12 per cent on the three coarser sizes of gravel. Incidentally, there is no apparent explanation for the high percentage of breakage in the intermediate gravel during the first two months of operation. The few tests recorded here might have been misleading, but the contractor, in taking a much larger number of tests, obtained similar values. In order to show only the undersize that was the result of breakage in transit, the data for Fig. 2 were computed by subtracting the undersize at the aggregate plant from that at Coram and the mixing plant, then expressing this difference as a per cent of the perfectly graded portion of the material leaving the aggregate plant. Discrepancies, such as apparently negative breakage, are due largely to the difficulties encountered in obtaining representative samples of segregated aggregate from a moving conveyor or discharging bin, and the impracticability of attempting to sample identical gravel as it travels along the conveyor or through the stockpiles.

An observation of the original transfer in operation gave the impression that the chutes were receiving their load nicely, the upper end being only an inch or two below the gravel trajectory. However, the load velocity increased considerably in the chute, causing gravel breakage, and abrasion of the receiving belt. The chute liners, which were of ½-in. manganese steel, were worn through before 250,000 tons of aggregate had been delivered. Some aggregate speeds in the chutes were tested with a speedometer on a bicycle wheel and were found to be about 1000 ft. per minute. This test, however, was very crude, and gave a wide variation of results.

Fig. 1 with its sketches 1 to 9, shows several chutes tried. The main difficulty was to find a chute which would slow the gravel but not plug while delivering sand. The grizzly idea shown at 2 appeared plausible, as it would force the larger sizes to slide farther than the small ones. However, a grizzly of sufficient length to slow a rolling rock would stop a flat sliding rock, and plug the chute. All of the other ideas were discarded after one or two trials. Finally the transfer shown at 8, developed at Grand Coulee, was adopted. Installation of this type of transfer was completed in October, 1940, and subsequent breakage has been relatively small.

The type shown at 9 is used by the dam contractor. That conveyor system contains 10 such transfers, two hoppers, and one stock pile. It was believed that most of the breakage in this system was caused by stockpiling, rather than by the transfers. However, since the Redding-Coram conveyor was constructed with 10-ft. drops between flights, the adoption of this type of transfer would have entailed considerable expense.

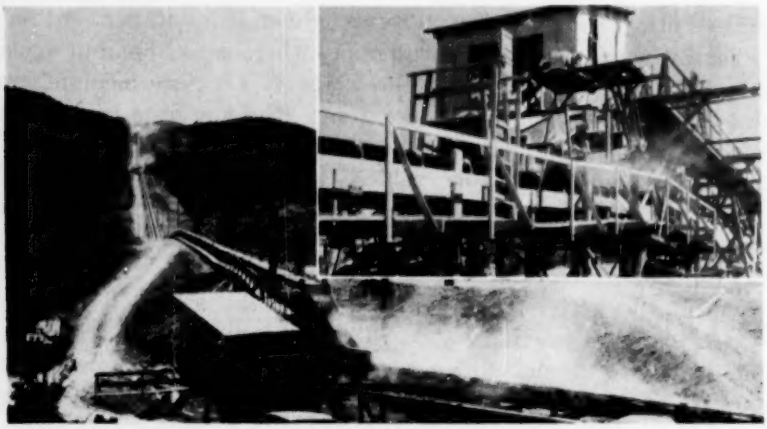

Fig. 4—Last four flights of conveyor, showing original transfer chute at 1 in Fig. 1
Fig. 5 (above)—An experimental transfer chute sketched at 6, in Fig. 1

Fig. 3 shows grading curves on the gravel at the Redding gravel plant, the Coram hoppers, and the mixing plant. These tests were made in August, 1940, when the original transfers were being used. The purpose of the tests was to determine the grading of the undersize. The results show that the gravel was not shattered, but was reduced somewhat in size by successive chipping. These chips collected at the bottom of drawdown cones in the hoppers and stockpiles, then came to the mixing plant in slugs. Therefore it is believed that the breakage between the dam stockpiles and the mixing plant is not as troublesome as that which takes place along the Redding-Coram conveyor.

It is interesting to note that the sand loses some fines in transit. Some of this material adheres to the belt, then is brushed off by the return rollers, forming mounds of stalagmitic appearance on the ground below them. The average for about the first 1000 samples at each place shows average fineness moduli of 2.88, 2.95 and 2.99 at the aggregate plant, Coram, and the mixing plant, respectively.

All aggregates accumulate some rock dust in transit. This sometimes amounts to as much as one per cent. In the summer, the material is sprinkled with water while moving along the conveyor. This tends to keep the dust down, the absorption up, and the belt cool. If the material is allowed to become too dry the air at the transfers becomes very dusty; but if it is kept damp a coating builds up on the gravel. However, occasional sprinkling, allowing time to dry between sprays, gives fairly clean gravel.

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The Effect of Change in Moisture-Content of the Creep of Concrete Under a Sustained Load*

BY GERALD PICKETT†

SYNOPSIS

The amount and rate of plastic flow in concrete under load has been found to depend upon the rate of drying. This paper shows by mathematical analysis that this is a natural consequence of non-uniform shrinkage and a non-linear stress-creep relationship. It further shows that shrinkage cannot account for additional creep unless inelastic strain not proportional to stress is produced. Results from experiments designed to test the applicability of the theory to concrete are reported.

INTRODUCTION

The plastic flow or creep of concrete under load has been studied for many years. The investigation by R. E. Davis and his associates at the University of California,[1][2] beginning in 1925 is perhaps the most extensive. In general, constant axial loads were applied to cylinders and the deformations with time were compared with those of companion cylinders not loaded. Some of these specimens remained loaded for more than seven years. The effects upon the time-flow relation were studied, of such variables as composition of the concrete, age when loaded, magnitude of stress, repeated loading, and the humidity of the air in which the specimens were stored. The conclusion was reached[1] that

> . . . the flow of plain concrete under a constantly sustained compressive stress (1) continues to increase, at first rapidly and then more slowly, for a long time, certainly for several years, (2) is greater for concrete in an atmosphere of low humidity than for an identical concrete in an atmosphere of high humidity or when fully saturated with water, (3) is materially affected by the character of the aggregate, being substantially larger for sandstone concrete than for limestone concrete, (4) is less for rich mixes than for lean ones, (5) is less for concrete having a high modulus of elasticity than for one having a low modulus of elasticity, (6) is greater for a high water-

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†Portland Cement Assn., Chicago.

cement ratio than for a low one, and (7) is greater when load is applied at an early age than when it is applied at a later age.

Their conclusions in regard to the effect of magnitude of stress on creep are not so definite.[2] They state that under their conditions of test the flow is roughly proportional to sustained stress for stresses from 300 to 900 psi but that the relation does not appear to hold outside these limits.

Glanville, Building Research Station, England,[3] agrees with Davis on most points. Glanville found that creep is approximately proportional to stress for stresses up to about ¼ to ⅓ of the ultimate strength. Also, he tested cylinders in tension and concluded that creep is the same for compressive and tensile stresses of the same magnitude. He used an average stress of 150 psi and did not expect this relationship to hold for stresses close to the ultimate tensile strength.

Creep in beams[3] and columns[4][5] both plain and reinforced has been studied. The results for the most part are in accord with the above conclusions.

During the past decade most investigators have emphasized the large effect of storage conditions (chiefly humidity) upon creep. But they are not agreed as to whether this is a direct effect or the result of another action which depends upon humidity. The investigations of both Glanville and Davis were conducted under controlled conditions of humidity and temperature although (as may be seen from Fig. 5 of Ref. 3) control was not always as good as desired. A lack of control of these factors may account for some contradictory results from other sources. Davis found that specimens stored in air at 50 per cent relative humidity flowed three times as much as specimens stored in water. In his analysis he considered the increase of deformation with time to consist of shrinkage due to drying and of flow due to load. He measured the shrinkage of companion unloaded specimens and by subtraction determined the flow or creep due to load. Some other investigators do not agree with this method of separating the deformations into shrinkage and flow. Lynam[6] says that "Shrinkage and creep are two aspects of the same phenomenon; it is impossible to separate them and no good end is served by trying to do so". Lorman[7] says that "Shrinkage or swelling due to loss or gain of moisture and creep due to seepage are interrelated phenomena". Lorman, however, considers the effects of the two separately. Maney[8][9] says that nonuniform shrinkage is the "source of all time-yield in loaded concrete members" and that this time-yield (flow as defined by Davis) is not plastic flow but is "merely an elastic redistribution of stress, increasing with time".

The apparent confusion and disagreement that has arisen is due in part to the lack of adequate theoretical analysis of the state of stress and strain in a body such as concrete which shrinks when it dries and yields under sustained stress. In general, stress has been calculated from the elementary formulas $\sigma = P/A$ and $\sigma = My/I$ without due consideration to the fact that non-uniform shrinkage may produce a distribution of stress greatly different from that given by elementary formulas. The unit deformation (resultant of elastic strain, shrinkage-strain, creep-strain, etc.) of each differential element of volume of concrete should depend upon actual conditions of stress, moisture content, etc., *in that particular element.* An analysis should deal with stress, strain, shrinkage, creep, etc., as functions of the coordinates of the specimen as well as of the usually considered variables, time, loading, storage, mix, etc. It is possible that the apparent effect of drying upon creep is not to increase the stress-creep coefficient but to cause non-uniform volume-changes that produce transient stresses which combine with the load-stresses and give rise to a distribution and magnitude of stress far different from that in a companion specimen undergoing less drying-shrinkage. A hypothesis based on the assumption that concrete has a non-linear stress-creep relation is sufficient to explain the observed difference in creep under different storage conditions.

This paper will show, by the solution of certain hypothetical problems, the necessary consequences of making various assumptions, and will demonstrate the mathematical procedure involved. The results of these analyses will be compared with the results obtained from actual specimens, treated in such a way as to duplicate as closely as possible the conditions assumed for the mathematical analyses. The paper does not deal with the mechanism by which creep (yielding under sustained stress) takes place. Such inelastic strain produced by stress may be the result of crystalline flow, viscous flow, seepage, or any combination of these. An important conclusion of the paper is that *an increase in creep accompanying non-uniform shrinkage or swelling is a natural consequence of the fact that the sustained-stress-vs.-strain curve for concrete is not linear.*

THEORETICAL ANALYSIS

Two hypothetical materials

In the following analysis two hypothetical materials will be considered. These materials are considered to be homogeneous and isotropic and to be of such nature that, after having been subjected to *sustained* stresses, one will show a linear relation between stress and accumulated strain, and the other, a non-linear relation. In this analysis,

as in some structural problems, a separation of deformation due to stress into its elastic and inelastic components is not necessary; only the sustained-stress-vs.-strain relation for the material need be given. However, since the utility of the analysis in interpreting the behavior of concrete depends upon the similarity of the properties assumed for the hypothetical materials to those of concrete, the following observations, applicable to concrete, are in order:

(1) A linear relation between sustained stress and strain implies that creep *if present* is proportional to stress.

(2) A non-linear relation implies that creep *is* present and is not proportional to stress.

(3) In so far as creep occurs, the sustained-stress-vs.-strain relation changes with the period of loading, that is, the relation is necessarily a function of time. It may also depend upon other factors which finally make it a function of the space-coordinates as well as of time.

It will be assumed that rectangular prisms of these hypothetical materials are subjected to various loading conditions and that while some of the prisms do not shrink, others do in a definite way. Any of the distributions of shrinkage-strain that might exist in concrete would serve the purpose of this analysis if shrinkage-stresses were produced, but the mathematical work is simplified by making the variation a simple function of one coordinate. Such an assumed distribution of shrinkage corresponds to the distribution in a rectangular prism drying from only one surface or from two opposite surfaces. Although accurate representation of shrinkage-strain as a function of coordinates and time would involve complicated expressions, there is justification for assuming, after the initial stages of drying, a parabolic distribution of the type

$$s = A\left(\frac{y}{a}\right)^2 + B$$

for drying from the top and bottom surfaces. A and B are functions of time, $2a$ is the depth of the prism, and s is unit linear shrinkage-strain.

The mathematical work is further simplified by assuming that the prism is long and narrow so that only longitudinal stresses and strains need be considered. However, it is believed that the effect of width upon the longitudinal stresses and strains would be small.

The cases to be analyzed in detail are given in Table 1.

TABLE 1—RELATIONSHIP BETWEEN SUSTAINED STRESS AND STRAIN

Stress-Strain Relation	Shrinkage-Strain $s = -\left[3\left(\frac{y}{a}\right)^2 + 2\right] \times 10^{-4}$			No Shrinkage	
	No Load	Tensile Load 500 psi	Moment Load $333a^2$ in-lb/in	Tensile Load 500 psi	Moment Load $333a^2$ in-lb/in
Non-linear $\sigma = 500 \tan^{-1}(e \times 10^4)$	Case I	Case II	Case V	Case III	Case IV
Linear $\sigma = 5 \times 10^6 e$	Case VI	Case VII	Case X	Case VIII	Case IX

σ = stress; e = strain due to stress (elastic + creep); s = shrinkage-strain

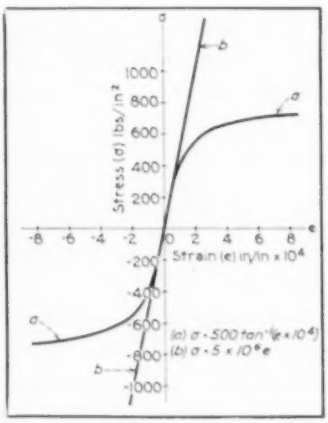


Fig. 1—Hypothetical sustained-stress-vs.-strain relations some time t after load is applied

Stress not proportional to strain (cases I to V)

Consider a hypothetical material having, after some period of loading, the sustained-stress-vs.-strain (σ, e) relation as given by Equation 1.

$$e = 10^{-4} \tan \frac{\sigma}{500} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

or, solving for σ,

$$\sigma = 500 \tan^{-1} (e \times 10^4) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1a)$$

This equation is represented by curve a in Fig. 1. The curve shows that a stress of, say, 400 psi would produce a strain of about 0.0001 in/in, and—note especially—it shows that this strain could be doubled by increasing the stress only 160 psi. It would not be possible to add a second stress of 400 psi to make a total of 800 psi since 785 psi would produce infinite strain (rupture). This shows that with such a stress-

strain relation superposition of stresses does not result in superposition of strains and vice versa. Therefore, it may be concluded that a stress-producing shrinkage would cause an effect in a body under load different from that which the same shrinkage would cause in a companion body not under load. That there would be a difference appears obvious, but the specific manner in which the effects of shrinkage differ in the two bodies is not easily determined. The difficulty in the analysis arises from the fact that although the loading of a specimen and the distribution of shrinkage may be arbitrarily assigned, the distribution of the resultant stresses or resultant strains within the body cannot be arbitrarily assigned. Not only must the actual distributions be in accord with the stress-strain characteristics of the material but also the distributions of stress and strain are controlled by the requirements of equilibrium and compatibility, respectively[10].

The special case in which the material has a linear stress-strain relation is treated in texts on the theory of elasticity, usually under the subject of thermal stresses. So far as it is known, the non-linear case, here analyzed, has not been treated before.

It is emphasized that the stresses here discussed are the actual stresses at a point and are not the conventional stresses calculated by elementary formulas; and the strains under discussion are also the strains at a point. Moreover, a strain at a point may be the resultant of several kinds of strains. The various kinds of strains are defined below.

Shrinkage-Strain (s) (Swelling-strain if positive) is that unit deformation due to any cause other than stress which would occur in an infinitesimal element if the element were unrestrained by neighboring elements. It is not to be confused with the *average* unit deformation, commonly called shrinkage of a so-called unrestrained specimen.

Creep-Strain (c) is inelastic unit-deformation of an infinitesimal element due to stress. It is not to be confused with the term creep, which denotes inelastic deformation of the body as a whole due to loads.

Strain Due to Stress (e) is the sum of the elastic-strain and creep-strain in an infinitesimal element. For simple tension or compression,

$$e = \frac{\sigma}{E} + c \qquad (2)$$

It follows that if stress is given by Equation 1a, creep will be given by Equation 2a:

$$c = 10^{-4} \times \tan(\sigma/500) - \sigma/E \qquad (2a)$$

and therefore the stress-creep relation will not be linear.

Total, or Resultant, Strain (e') is the resultant unit-deformation:

$$e' = e + s$$

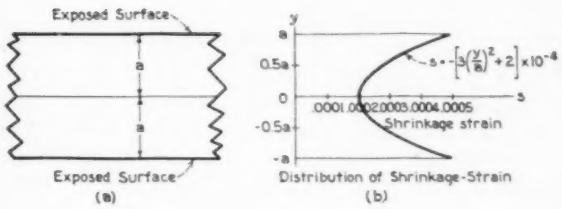


Fig. 2—(a) left; (b) right

Shrinkage alone (case I)

Case I pertains to a prism drying from two opposite surfaces, as shown in Fig. 2a. It is assumed that some time after drying begins the distribution of shrinkage-strain is that given by Equation 4.

$$s = -\left[3\left(\frac{y}{a}\right)^2 + 2\right] \times 10^{-4} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

This relationship is shown in Fig. 2b.

It is further assumed that sections remain plane. This latter assumption is essentially correct for sections not close to an end as can be reasoned from Saint Venant's principle,[11] or it can be proved by the principle of minimum energy. If sections remain plane, then it can be shown that the conditions for compatibility are satisfied. In so far as the sections remain plane and shrinkage is symmetric with respect to a horizontal plane through the longitudinal axis, then the resultant strain (e') is independent of y, or,

$$e' = \text{constant} = s + e \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Substitution of Equation 5 into Equation 1a gives,

$$\sigma = 500 \tan^{-1} (e' - s) \times 10^4 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

Equilibrium requires that

$$\int_{-a}^{+a} \sigma dy = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

Combining Equations 4, 6 and 7 the result becomes

$$500 \int_{-a}^{+a} \tan^{-1}\left[e' \times 10^4 + 3\left(\frac{y}{a}\right)^2 + 2\right] dy = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

Equation 8, solved by trial, gives a value for e' of approximately -2.929×10^{-4} in./in. Under the conditions assumed this is the resultant strain. Unfortunately, it is commonly called shrinkage and incorrectly used as if it were something apart from elastic or plastic deformation.

The shrinkage-stress is now found by substituting for e' and s in Equation 6.

$$\sigma = 500 \tan^{-1}\left[3\left(\frac{y}{a}\right)^2 - 0.929\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

This distribution is shown in Fig. 3.

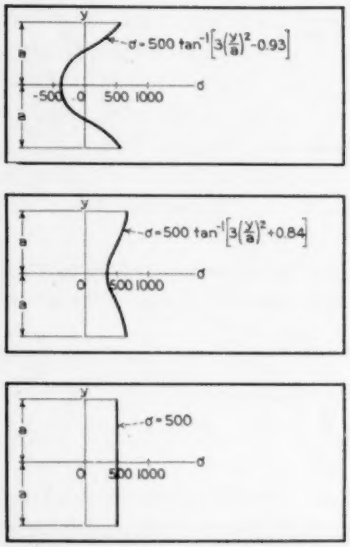


Fig. 3—Distribution of stress for shrinkage (case I)

Fig. 4—Distribution of stress for tension and shrinkage (case II)

Fig. 5—Distribution of stress for tension (case III)

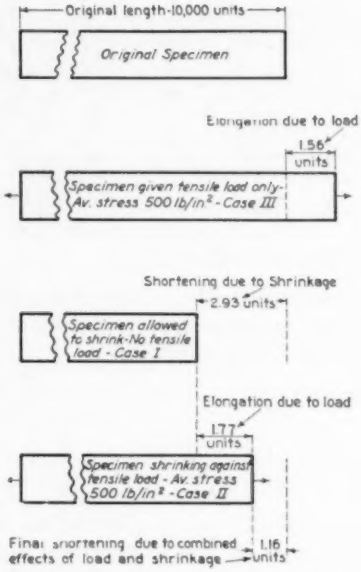

Original length-10,000 units
Original Specimen
Elongation due to load
1.56 units
Specimen given tensile load only-Av. stress 500 lb/in² - Case III
Shortening due to Shrinkage
2.93 units
Specimen allowed to shrink-No tensile load - Case I
Elongation due to load
1.77 units
Specimen shrinking against tensile load - Av. stress 500 lb/in² - Case II
Final shortening due to combined effects of load and shrinkage
1.16 units

Fig. 6—Diagrammatic summary of the analyses of cases I, II, and III

Tensile load and shrinkage (case II)

Case II differs from Case I in that, in addition to the shrinkage-strain, a load is assumed to be acting which, by elementary calculation, would give a uniform tensile stress of 500 psi.

Under this load equilibrium requires that

$$\int_{-a}^{+a} \sigma \, dy = 500 \times 2a \qquad (7a)$$

Combining Equations 4, 6 and 7a the result becomes

$$500 \int_{-a}^{+a} \tan^{-1}\left[e' \times 10^4 + 3\left(\frac{y}{a}\right)^2 + 2 \right] dy = 1000a \qquad (8a)$$

Equation 8a is approximately satisfied by a value of $e' = -1.163 \times 10^{-4}$ in/in.

The equation for resultant stress becomes

$$\sigma = 500 \tan^{-1}\left[3\left(\frac{y}{a}\right)^2 + 0.837 \right] \qquad (9a)$$

This distribution is shown in Fig. 4.

Tension without shrinkage (case III)

Case III differs from Case II only in that shrinkage-strain is not present. Therefore, $e' = e$. Because of the absence of shrinkage a uniform tension applied to a prism produces uniform stress throughout; and, in such a simple case, the strain may be found directly from the stress-strain relation. For $\sigma = 500$ psi this strain is given by Equation 1 to be $e' = +1.557 \times 10^{-4}$ in/in. This uniform distribution of stress is shown in Fig. 5.

Summary of cases I, II, and III

The results of the above analysis are shown in Table 2 and graphically in Fig. 6.

TABLE 2

Case	Load	Shrinkage-Strain	Average Unit Deformation (Ten-Thousandths)
I	No load	$s = -\left[3\left(\frac{y}{a}\right)^2 + 2\right] \times 10^{-4}$	−2.929
II	Tensile load 500 psi	$s = -\left[3\left(\frac{y}{a}\right)^2 + 2\right] \times 10^{-4}$	−1.163
III	Tensile load 500 psi	None	+1.557

Since the assumed sustained-stress-vs.-strain relation is regarded to be a function of time, the elongation shown in Fig. 6 for Case III is to be considered the result of both elastic and creep elongations. That is, the 1.56 units of elongation shown represent the sum of the elastic and creep elongations of a specimen 10,000 units in length, due to a sustained tensile load of 500 psi. The difference between the calculated shortenings under the conditions of Cases I and II is considered to be an elongation due to load since the specimens are assumed to be otherwise identical. The calculations show that this elongation due to load is 1.77 units whereas in Case III the value is only 1.56 units. Suppose the elastic part of the elongation in these cases is 1.0 units, thus leaving 0.77 units for creep when shrinkage is present and 0.56 when shrinkage is not present. An experimenter obtaining such results from a material like this hypothetical one might conclude that the stress-creep relation was affected by drying. This conclusion would be incorrect for such a material, because, as has now been mathematically demonstrated, an increase in creep accompanying non-uniform shrinkage is a natural consequence of the non-linear stress-strain relation of the material.

Similar considerations for flexural stresses follow:

Moment load without shrinkage (case IV)

Case IV represents a prism loaded to produce flexure. The prism does not have shrinkage-strain but is assumed to have the non-linear sustained-stress-vs.-strain relation given by Equation 1. If stress (σ) and curvature ($1/\rho$) were calculated by the formulas of elementary mechanics, $\sigma = \frac{My}{I}$ and $\frac{1}{\rho} = \frac{M}{EI}$, the calculations would show,

assuming a moment-load of $333a^2$ in-lb/in of width, the maximum stress to be 500 psi and the curvature to be $\frac{500}{Ea}$ in^{-1}. However, these formulas do not apply, since only strain and not stress is proportional to distance from the neutral axis.

The relation for strain in this case is:

$$e' = \frac{y}{\rho} \quad \ldots\ldots (10)$$

From consideration of equilibrium

$$\int_{-a}^{+a} \sigma y \, dy = M$$

or

$$500 \int_{-a}^{+a} y \tan^{-1} (e \times 10^4) \, dy = 333a^2 \quad \ldots\ldots (11)$$

Since $s = 0$, $e' = e$; therefore, from Equations 10 and 11

$$500 \int_{-a}^{+a} y \tan^{-1} \left[\frac{y}{\rho} \times 10^4 \right] dy = 333a^2 \quad \ldots\ldots (12)$$

The curvature $\frac{1}{\rho}$ is found by trial to be $\frac{1.236 \times 10^{-4}}{a}$ in^{-1}. The resultant stress is found by substituting in Equation 1*a* to be,

$$\sigma = 500 \tan^{-1} \left(1.236 \frac{y}{a} \right) \quad \ldots\ldots (9b)$$

This distribution is shown in Fig. 7.

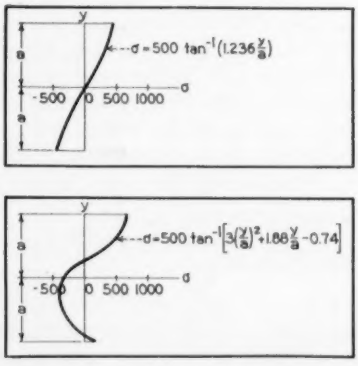


Fig. 7—Distribution of stress for bending (case IV)

Fig. 8—Distribution of stress for bending and shrinkage (case V)

Moment load and shrinkage (case V)

Case V is a combination of Case IV (loaded in flexure) and Case I (shrinkage-strain). It represents a prism loaded to produce flexure, drying from the two surfaces where the maximum load-stresses would occur. The relation for strain in this case is,

$$e' = \frac{y}{\rho} + k \qquad (13)$$

where k is the average unit change in length. This k will be of the same order of magnitude but not identical with the unit change in Case I which was -2.929×10^{-4}.

From Equations 3, 4 and 13

$$e = \frac{y}{\rho} + k + \left[3\left(\frac{y}{a}\right)^2 + 2\right] \times 10^{-4} \qquad (14)$$

From consideration of the conditions of equilibrium

$$\int_{-a}^{+a} \sigma dy = 0 \qquad \text{and} \qquad \int_{-a}^{+a} \sigma y\, dy = M$$

From these expressions and Equation 1, it follows that

$$500 \int_{-a}^{+a} \tan^{-1}\left[3\left(\frac{y}{a}\right)^2 + 2 + \left(\frac{y}{\rho} + k\right) \times 10^4\right] dy = 0 \qquad (15)$$

and,

$$500 \int_{-a}^{+a} y \tan^{-1}\left[3\left(\frac{y}{a}\right)^2 + 2 + \left(\frac{y}{\rho} + k\right) \times 10^4\right] dy = 333a^2 \qquad (16)$$

The curvature $\frac{1}{\rho}$ and the average unit change in length are found by trial from the above equations to be:

$$\frac{1}{\rho} = \frac{1.881}{a} \times 10^{-4} \text{in}^{-1} \qquad \text{and} \qquad k = -2.742 \times 10^{-4} \text{ in/in}$$

Substitution of these values into Equation 1a gives

$$\sigma = 500 \tan^{-1}\left[3\left(\frac{y}{a}\right)^2 + 1.881\,\frac{y}{a} - 0.742\right] \qquad (9c)$$

This distribution is shown in Fig. 8.

SUMMARY OF CASES I, IV AND V (TABLE 3)

The curvature for Case IV, $1.24/a$ ten-thousandths, represents the effects of elasticity and creep. Since the shrinkage is symmetrical with respect to the central plane, shrinkage acting alone does not cause curvature. However, when shrinkage and load are both acting as in Case V, the curvature is $1.88/a$ ten-thousandths. An experimenter working with specimens of a material like this hypothetical one would be justified in concluding that the increase in curvature of Case V over that of Case IV was due to shrinkage. But to explain it as an effect of drying upon the stress-creep coefficient

TABLE 3

Case	Load	Shrinkage-Strain	Average Unit Deformation (Ten-Thousandths)	Curvature (Ten-Thousandths)
I	No load	$s = -\left[3\left(\frac{y}{a}\right)^2 + 2\right] \times 10^{-4}$	−2.929	Zero
IV	Moment-load $333a^2$ in-lb/in	None	Zero	$\frac{1.236 \text{ in}^{-1}}{a}$
V	Moment-load $333a^2$ in-lb/in	$s = -\left[3\left(\frac{y}{a}\right)^2 + 2\right] \times 10^{-4}$	−2.742	$\frac{1.881 \text{ in}^{-1}}{a}$

would be incorrect for, as has been shown, additional curvature accompanying shrinkage is a natural consequence of the non-linear stress-strain relation and not, in this case, the result of any effect of drying upon this relation.

Stress proportional to strain (cases VI to X)

In the preceding section it was shown that when the sustained-stress-vs.-strain relationship is not linear, additional creep is a natural consequence of the assumed shrinkage-strain represented by Equation 4. In this section it will be shown that if stress is proportional to strain and if the physical properties of the material (modulus of elasticity, etc.) remain unchanged, then shrinkage-strain can have no effect upon creep as ordinarily defined. The distinction between creep, which is inelastic deformation of the body as a whole caused by loads, and creep-strain, which is inelastic unit-deformation of a differential volume element, is again emphasized. Since shrinkage-strain or any other self-strain of itself does not limit the validity of the principle of superposition,[12] the independence of shrinkage and loads for bodies which follow Hooke's law should be obvious. However, it must be admitted that the properties of a material like concrete might not remain unchanged during the period of drying. It is conceivable that a change in moisture content, affecting hydration, strength, viscosity, or some other property, would so change the physical constants that additional creep could accompany shrinkage by virtue of this change regardless of the linearity of the stress-strain relation. Nevertheless, in this analysis of a hypothetical material it is assumed that the physical properties do remain constant.

Since strain due to stress includes both elastic strain and creep-strain ($e = \sigma/E + c$) a linear relation of stress to strain would be found if creep-strain (c) were proportional to stress and the instantaneous modulus (E) were independent of stress. Lorman[7], Shank[4] and others have made these assumptions in their equations for creep of concrete. Lorman says, "Results of the more recent laboratory tests point to the conclusion that within the range of design stresses, creep-strain or unit creep is proportional to the stress." Maney even eliminates creep from consideration and says,[9] "Concrete could be regarded as a truly elastic material for all working load or design conditions." It should be noted that either of these assumptions implies that the sustained-stress-vs.-strain relationship is linear. The following analysis will show that if stress is proportional to strain, and if the properties of the material are not changed by drying, shrinkage can have no effect on creep. Because this has heretofore been overlooked, the analysis seems justified, even though in essence it merely verifies the principle of superposition.

Consider a hypothetical material with the linear sustained-stress-vs.-strain relation given by Equation 17.

$$\sigma = 5 \times 10^6 e \quad \ldots\ldots (17)$$

This straight-line relation is shown by curve *b* in Fig. 1. Note that the figure shows that at the origin this curve is tangent to the curve representing the non-linear stress-strain relation of the other hypothetical material. Therefore, both assumed materials would have approximately the same characteristics under the same condition of loading and shrinkage-strain as long as the resultant stresses were small.

Shrinkage alone (case VI)

Case VI is the same as Case I in that it represents a prism drying from two opposite surfaces. It differs from Case I only in that stress (σ) and strain (e) are assumed to be proportional. Equations 4, 5 and 7 of that analysis apply here, but Equations 6 and 8 are replaced by Equations 18 and 19 as follows:

Substituting Equation 5 into Equation 17 gives

$$\sigma = 5 \times 10^6 (e' - s) \quad \ldots\ldots (18)$$

Combining Equations 4, 18 and 7 the result becomes

$$5 \times 10^6 \int_{-a}^{+a} \left(e' + \left[3 \left(\frac{y}{a} \right)^2 + 2 \right] \times 10^{-4} \right) dy = 0 \quad \ldots\ldots (19)$$

from which e' is found to be -3×10^{-4} in/in. This unit shortening is to be compared with $e' = 2.929 \times 10^{-4}$ in/in found in Case I.

Substituting $e' = -3 \times 10^{-4}$ in/in into Equation 8 gives

$$\sigma = 500 \left[3 \left(\frac{y}{a} \right)^2 - 1 \right] \quad \ldots\ldots (20)$$

for the shrinkage stress.

Tensile load and shrinkage (case VII)

This differs from Case VI in that an average tensile load is also acting during drying. It differs from Case II only with respect to the assumed sustained-stress-vs.-strain relation.

Equilibrium is satisfied by Equation 7a.

Combining Equations 4, 18 and 7a the result becomes

$$5 \times 10^6 \int_{-a}^{+a} \left(e' + \left[3 \left(\frac{y}{a} \right)^2 + 2 \right] \times 10^{-4} \right) dy = 1000a$$

from which e' is found to be -2×10^{-4}in/in. This average unit shortening is to be compared to -1.163×10^{-4}in/in found for Case II.

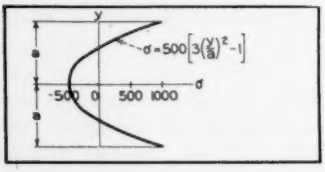
Tension and moment without shrinkage (cases VIII and IX)

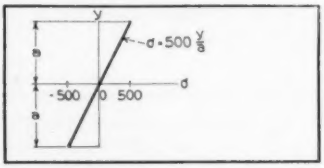
The elementary formulas apply in these cases. For Case VIII the unit strain

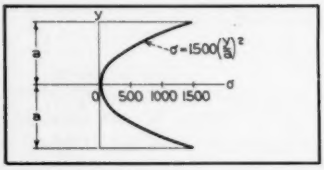
$$e' = e = \frac{500}{5 \times 10^6} = 10^{-4} \text{ in/in}$$

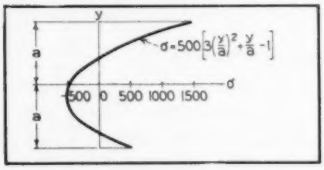
For Case IX the curvature

$$\frac{1}{\rho} = \frac{M}{EI} = \frac{333a^2}{5 \times 10^6 \times 2/3\, a^3} = \frac{10^{-4}}{a} \text{ in}^{-1}$$









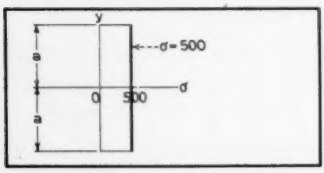


Fig. 9 (top left)—Distribution of stress for shrinkage (case VI)

Fig. 10 (center left)—Distribution of stress for tension and shrinkage (case VII)

Fig. 11 (bottom left)—Distribution of stress for tension (case VIII)

Fig. 12 (top right)—Distribution of stress for bending (case IX)

Fig. 13 (bottom right)—Distribution of stress for bending and shrinkage (case X)

Shrinkage and moment (case X)

Case X is the same as Case V in that it represents a prism under flexural load, drying from two opposite surfaces, and differs only in the assumed stress-strain relation. Equations 3, 4, 13 and 14 of that analysis apply here, but Equations 15 and 16 are replaced by

$$500 \int_{-a}^{+a} \left[3 \left(\frac{y}{a} \right)^2 + 2 + (y/\rho + k) \times 10^4 \right] dy = 0 \quad \ldots\ldots\ldots\ldots (20)$$

$$500 \int_{-a}^{+a} y \left[3 \left(\frac{y}{a} \right)^2 + 2 + (y/\rho + k) \times 10^4 \right] dy = 333a^2 \quad \ldots\ldots\ldots (21)$$

The curvature $1/\rho$, and the unit change in length, k, are found from these equations to be

$$\frac{1}{p} = \frac{10^{-4}}{a} \text{ in}^{-1} \qquad \text{and} \qquad k = -3 \times 10^{-4} \text{ in/in}$$

The distributions of stresses for the last five cases are shown in Fig. 9 to 13.

Summary of cases VI to X

Table 4 shows the results of the analysis when the sustained-stress-vs.-strain relation is assumed to be linear.

The curvature due to flexural load without shrinkage (Case IX) is the same as the curvature due to flexural load with shrinkage (Case X) showing that shrinkage has no

TABLE 4

Case	Load	Shrinkage-Strain	Average Unit Deformation (Ten-Thousandths)	Curvature (Ten-Thousandths)
VI	None	$s = -\left[3\left(\frac{y}{a}\right)^2 + 2\right] \times 10^{-4}$	−3.00	Zero
VII	Tensile load 500 psi	$s = -\left[3\left(\frac{y}{a}\right)^2 + 2\right] \times 10^{-4}$	−2.00	Zero
VIII	Tensile load 500 psi	None	+1.00	Zero
IX	Moment load $333a^2$ in-lb/in	None	Zero	$\frac{1.00}{a}$ in^{-1}
X	Moment load $333a^2$ in-lb/in	$s = -\left[3\left(\frac{y}{a}\right)^2 + 2\right] \times 10^{-4}$	−3.00	$\frac{1.00}{a}$ in^{-1}

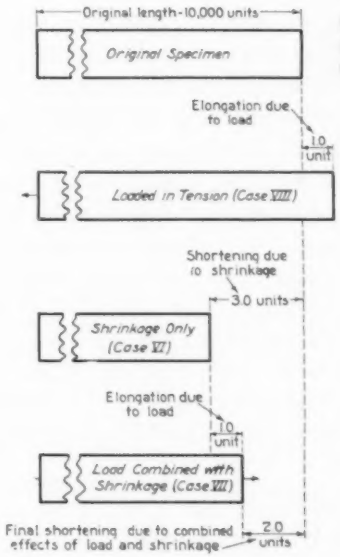


Fig. 14 — Diagrammatic summary of the analyses of cases VI, VII and VIII

effect on deflection when strain is proportional to stress. The final average shortening when both shrinkage and load are acting (Case VII) is the algebraic sum of the unit changes in length when these factors act separately (Cases VI and VIII). Or, as is better illustrated in Fig. 14, the load causes the same elongation when shrinkage is absent as when shrinkage is present.

Therefore, the principle of superposition is valid for stresses and strains arising from loads and shrinkage if stress is proportional to strain and if the proportionality is not affected by other factors (water-content, viscosity, etc.) which may be associated with shrinkage.

The mathematical work of this paper may be summarized as follows:

(1) If unit deformation is not proportional to sustained stress, then the sum of the separate effects of non-uniform shrinkage and of load acting successively is not the same as the effect produced by these acting simultaneously.

(2) If unit deformation is proportional to stress for all stresses involved, then the principle of superposition applies.

(3) The method of determining the combined effects of shrinkage and load for assumed conditions is explained and illustrated. However, *for other conditions* the method might need to be revised. For example, the stress-strain relation of each element of concrete probably depends upon its past history of stress; therefore, since different elements have had different stresses in the past, the relation would be a function of the space coordinates in addition to time.

APPLICATION TO CONCRETE

As previously stated, most investigators are agreed that the conditions of moisture surrounding the specimen are very important factors in creep of concrete. The above analysis has given one possible explanation for this effect. The applicability of this explanation to concrete depends upon the actual stress-creep relation possessed by concrete since the explanation requires a non-linear relation in the range of stresses involved. The investigations of Glanville and Thomas previously mentioned[3] give some information upon this relation. On the basis of their experiments they concluded that stress and creep were approximately proportional for low stresses but that for loads of from 36 to 84 per cent of the ultimate strength "creep is clearly not proportional to the applied stress over this range, but increases considerably as the ultimate strength is approached." These results dictated the choice of the particular non-linear stress-strain relation shown by curve *a* in Fig. 1.

Assuming this statement of the stress-creep relation for concrete to be essentially correct, it remains to be shown that the stresses under ordinary conditions of drying are of such magnitude as to be definitely in the non-linear range. The stresses found by elementary formulas when designing structures may be within the linear range, but actual stresses will depend upon the distribution of shrinkage. The fact that concrete frequently cracks is proof that stresses sometimes reach the ultimate strength in tension. If there is a considerable amount of creep just before the ultimate strength is reached, then it seems reasonable that stresses close to the ultimate are often reached without the formation of cracks since local yielding relieves local stresses. A de-

tailed calculation of the transient stresses caused by drying has been made by the author in unpublished work. The results show that under most conditions concrete would have severe cracking if it were not for creep. For example, if the surface of concrete has dried so that it would have a unit shortening of, say, 700×10^{-6} in/in if unrestrained, and if the elastic modulus were, say, 5×10^{6} psi, then, neglecting creep and assuming 50 per cent restraint (a conservative estimate for the surface-region of a concrete member during the first stages of drying), the tensile stress would be 1750 psi, i.e., far beyond the tensile strength of the concrete.

The above discussion indicates that actual stresses from the combined effects of load and shrinkage may be large enough in some regions of concrete members to be in the range where the stress-creep relation is definitely not linear. Therefore, it is believed that the analysis given provides an explanation for the observed influence of storage conditions on the behavior of concrete under load.

EXPERIMENTAL RESULTS*

Effect of drying

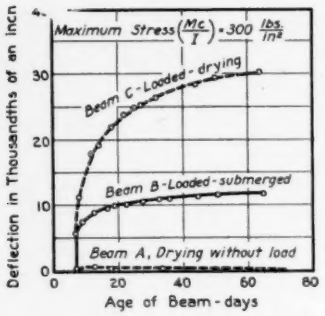
Fig. 15 shows the results from three beams which were tested under conditions comparable with those assumed for Cases I, IV and V.

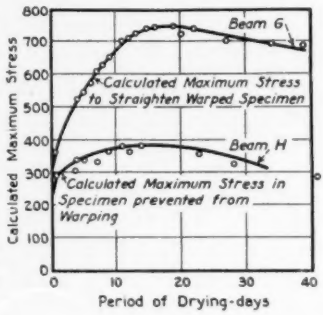
Beam A was allowed to dry without load. (The specimen was stored on end to prevent flexing from its own weight.) The negligible deflection demonstrates the symmetry of composition of the specimen. These conditions correspond to Case I.

Beam B, which is comparable with the hypothetical beam of Case IV, was loaded, but drying was prevented by submersion in water. Creep is shown by the increase in deflection with time.

Beam C, which is comparable with the hypothetical beam of Case V, had the same external load as beam B, and was subjected to the same humidity as beam A. After fifty days under load, the creep of C was four or five times the creep of B, and the total deflection was two and a half times the deflection of beam B. This is qualitatively in agreement with the theory, though the difference is greater than that accounted for in the theoretical discussion. The theoretical work could be changed so as to account for this much difference between beams C and B by the proper change in either the sustained-stress-vs.-strain relation assumed, Equation 1, or the assumed shrinkage-strain distribution, Equation 4. However, since other factors not taken into account in the above theory probably contribute to creep, this change has not been made.

*For details of materials and methods see Appendix.


Maximum Stress $\left(\frac{Mc}{I}\right)$ = 300 lbs./in²
Beam C - Loaded - drying
Beam B - Loaded - submerged
Beam A, Drying without load
Deflection in Thousandths of an inch
Age of Beam - days


Beam G
Calculated Maximum Stress to Straighten Warped Specimen
Beam H
Calculated Maximum Stress in Specimen prevented from Warping
Calculated Maximum Stress
Period of Drying - days

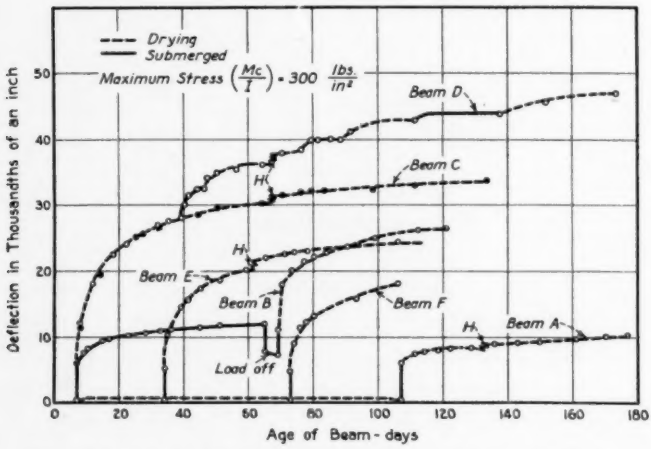


Fig. 15 (top left)—Effect of drying on the bending of loaded beams

Fig. 16 (above)—Deflections of loaded beams as influenced by gaun or loss in moisture content

Fig. 17 (top right)—Behavior of beams that are coated so that they will warp while they dry

The effects of additional treatment of these and other beams are shown in Fig. 16.

Effect of change in humidity and of alternate wetting and drying

After beam C, previously discussed, had been drying for about 60 days, a temporary failure of the air-conditioning equipment caused the relative humidity to lower from 50 per cent, its usual value, to 30 per cent. The effect on the deflection of the loaded beam is shown by the break in the curve designated by H. The humidity remained low only one day and then returned to normal. However, the increase in deflection of the beam was irreversible. This behavior, which occurred in all loaded beams that were drying at the time, may be accounted for as follows: The lowered humidity caused additional shrinkage at the surface. The additional surface-shrinkage caused tensile stresses at

the surface. These stresses are of the same sign as load stresses at the bottom of the beam but of opposite sign at the top of the beam. Therefore, owing to the non-linearity of the stress-creep relation, a relatively larger amount of creep occurred at the bottom of the beam without a corresponding diminution at the top of the beam, causing a much greater average creep-strain than would have occurred without the change in humidity. In so far as the action was due to creep and therefore inelastic, it was irreversible.

It is possible that lack of adequate control of humidity may account for some of the creep which others have reported to continue even after long periods of loading.

Observe the effects of alternate wetting and drying of beam D. Beam D was subjected to the same conditions as beam C for the first 28 days of drying. Then the beam was submerged in water without disturbing the load. A large increase in rate of creep occurred as shown. The beam was further subjected to alternate drying in air at 50 per cent relative humidity and submersion in water which produced additional irreversible creep with each cycle as shown.

After beam B, previously discussed, had been under load for 58 days while submerged in water, the load was removed. The amount of immediate elastic recovery indicated an increased modulus of elasticity. After four days, during which time there was a small decrease in deflection, the beam was reloaded and exposed to drying at 50 per cent relative humidity. The action of this beam while drying shows again the interaction of shrinkage and load. The creep which occurred during the period of drying was much more than that which occurred when drying was prevented. The fact that beam B did not creep as much during the period of drying as beams C and D may be attributed to the fact that beam B had cured under water for 69 days before drying began, whereas beams C and D had cured under water for only 7 days.

Effect of curing

The effect of prolonged wet-curing is further shown by the deflection of beams E and F, cured for 34 and 73 days, respectively, before being subjected to the normal loading and drying conditions. The behaviors of E and F, when compared with each other and with those of beams C and D, show that increased curing reduces both the elastic deflection and the creep. A comparison of the deflection of beam F with that of beam B during the period of drying indicates that the creep of beam B during the drying-period was not appreciably affected by the earlier loading that this beam had had during its period of curing under water.

Effect of drying before loading

After beam A, previously discussed, had been drying for 100 days and perhaps had reached a static condition with respect to moisture-content, the beam was loaded. The very small amount of creep shown indicates that shrinkage-stresses, if present, were so small as to have very little effect.

Magnitude of shrinkage-strain

Some auxiliary tests were made to obtain some indication of the magnitude of "free shrinkage" stresses and to determine whether such stresses produce appreciable creep. The tests were based on the fact that if a beam is allowed to dry from one side only, it will warp if free from external restraint. The warping will increase rapidly at first, then will slowly reach a maximum, after which it will slowly straighten, until finally the warping is less than one-half the maximum value. Since such a beam warps into an arc of a circle, it may be straightened by a moment-load. This moment may be measured experimentally, or from information as to stiffness and the amount of warping, it may be computed.

Two beams, G and H, were cured 7 days and then, after being covered on all sides but one, were allowed to dry. Beam G was allowed to warp. No external restraint or force was applied at any time, but the moment necessary to straighten the beam was computed.

Beam H was prevented from warping at all times, and the required moment was measured.

Fig. 17 shows the results of this experiment. The lower curve shows the stresses required to offset the tendency of the restrained specimen to warp, the stresses being expressed in terms of the formula $\sigma = Mc/I$. The upper curve shows, in the same terms, the stress that would be required to straighten the unrestrained specimen at any given time. The differences between stresses at a given time shown by the two curves are attributed to creep.

The maximum stress in the restrained specimen at the end of the first day of drying is shown to be 300 psi. The actual maximum stress was probably higher because of the stress-concentration at the exposed surface. It probably approached the ultimate strength of the material which, by auxiliary tension tests, was found to be 400 psi after one day of drying.

The opposite halves of a beam twice as thick as beam H and drying from two opposite sides would mutually provide restraint against warping and therefore be under the same conditions of stress and strain as beam H. Therefore, the results show that, even in so-called "free

shrinkage" prisms, the stresses may be close to the ultimate strength, and cracking would take place if it were not for creep.

CONCLUSION

Many factors other than those treated in this discussion must be considered before creep of concrete can be completely explained. The various factors may be so interrelated that the effects of any one can be only partially isolated. Certainly, changes in moisture-content do more than cause volume-changes. The properties of an agglomerate like concrete, usually undergoing spontaneous chemical and physical changes, cannot be represented in any great detail by a simplified, hypothetical material such as was treated above. Nevertheless, it is believed that the above theoretical and experimental results, together with the results of previous investigators, lead to the following rather definite conclusions:

(1) The existence of a non-linear sustained-stress-vs.-strain relation is enough to account qualitatively for an increase in creep of a body undergoing non-uniform shrinkage over that of a body not shrinking.

(2) Experimental evidence indicates that concrete has this non-linear relation, at least in the higher ranges of stress.

(3) High stresses are produced by ordinary drying, as is indicated by frequent cracking, and as was verified by measuring the magnitude of moment necessary to restrain one-half of a prism from warping.

Therefore, it is believed that the observed effect of changes in moisture-content on the creep of concrete under sustained load is due largely to a non-linear relation between the actual stresses and strains.

BIBLIOGRAPHY

1. Davis, R. E. and Davis, H. E.: "Flow of Concrete under the Action of Sustained Loads," ACI JOURNAL, Mar. 1931; *Proceedings* V. 27, p. 837.

2. Davis, R. E., Davis, H. E., and Hamilton, J. S.: "Plastic Flow of Concrete under Sustained Stress," Proc. Am. Soc. Testing Materials *34*, Part II, 384 (1934).

3. Glanville, W. H., and Thomas, F. G.: "Studies in Reinforced Concrete, IV. Further Investigations on the Creep or Flow of Concrete under Load," Building Research Tech. Paper No. 21 (1939).

4. Shank, J. R.: "The Plastic Flow of Concrete," Ohio State University Bull. No. 91 (1935).

5. Glanville, W. H.: "Studies in Reinforced Concrete, III. The Creep or Flow of Concrete under Load," Building Research Tech. Paper No. 12 (1930).

6. Lynam, C. G.: *Growth and Movement in Portland Cement Concrete*. Oxford University Press, 1934. P. 90.

7. Lorman, W. R.: "The Theory of Concrete Creep," Proc. Am. Soc. Testing Materials *40*, 1084 (1940).

8. Maney, G. A. and Lagaard, M. B.: "Stress Increases in Compressive Steel under Constant Load Caused by Shrinkage," ACI JOURNAL, June, 1940; *Proceedings*, V. 36 p. 541.

9. Maney, G. A. "Studies in Engineering—No. 1 'Warping' or Non-Uniform Shrinkage over Cross-Section Found to Be Source of All Time Yield in Loaded Concrete Members," Northwestern Technological Institute (1941).

10. Timoshenko, S. *Theory of Elasticity.* McGraw-Hill, 1934. Ch. 7.

11. *Ibid.*, p. 31.

12. Southwell, R. V.: *Theory of Elasticity.* Oxford University Press, 1936, p. 77.

APPENDIX

The data on the cements and aggregate used, as well as the details of the mix-proportions and the method of making and testing the specimens, were omitted from the main body of the paper because the characteristics of the particular concrete of which the specimens were made had no important bearing on the conclusions. That is, the results in this paper show in a qualitative way how concrete under load would react to wetting or drying. A variation in concrete, due to either composition or method of preparing the specimen, or variation in the test-procedure would produce results differing quantitatively from those shown in the paper. But it is believed that the general trend shown in Fig. 15 to 17 by the eight individual beams is characteristic of all concrete. As a matter of fact, the beams represented in the diagrams are in several instances only selections from a larger number that showed the same behavior.

Since some readers may desire more information on individual beams discussed in the paper, the following details are given:

TABLE A—MIX BY WEIGHT

Material	Parts by Weight	
	Beams A to F	Beams G and H
Water	0.50	0.42
Cement	1.00	1.00
Sand	2.08	1.28
Aggregate	2.42	1.82

Consistency

Slump 5 to 6 inches—remolding effort 30 to 35 jigs.

Materials

Cement: The cement in beams A to F was a mixture of four brands purchased in Chicago. The specific surface by the Wagner method was 1665 cm²/g.

The cement of beams G and H was a normal portland of 1200 cm²/g specific surface.

Aggregate: Sand and gravel were obtained from Elgin, Illinois. The gravel was screened to pass a ⅜" sieve and be retained on a No. 4 sieve.

The sand was graded as follows:

Sieve No.	Per Cent Passing
100	5
48	13
28	43
14	67
8	82
4	100

Procedure

With individual exceptions noted later, the procedure in preparing and loading these beams was as follows:

The concrete was mixed in a power-driven, open-tub mixer. Immediately after mixing, the concrete was cast in 2x2x34-in. steel molds and consolidated by light vibration by placing the molds on a small platform-type vibrator. Covers were fastened on molds but not made watertight. Then, molds and all were stored under water for one day. The molds were then removed and the specimens were again submerged until the age indicated on the diagrams in Fig. 15 to 17.

Loading

The beams were placed on a side as cast on a loading-bench, which provided a 32-inch span. The load consisted of the weight of the beam and a fifty-pound force at the center of the span.

Exceptions

Beam A was originally made and used for an investigation not discussed in the paper. It was cast in a watertight, wooden mold. After the usual light vibration a watertight cover was put on. The mold was then vibrated with the top down and finally vibrated on end. It was stored on end the first day while in the mold. At the end of the curing period the top and bottom surfaces as cast were sealed with paraffin. It was then stored on end in 50 per cent relative humidity for 100 days, after which it was loaded.

After *beam D* had been under load for 28 days, the 55-lb. central load was replaced with two 42-lb. loads at the quarter-points. This caused a different distribution of bending moment but caused practically no change in the deflection at the center or in the rate of creep. During the change in loading, the beam was placed in a pan so that it could be submerged in water, as explained in the paper, without further disturbance in the loading.

Beams G and H. At the end of the curing period beams G and H were sealed on all but one side with lacquer and paraffin. Beam G was not loaded but was permitted to warp. The deflection at the center of beam H was prevented by restraining-loads at the quarter-points.

Discussion of this paper should reach the ACI Secretary in triplicate by June 1, 1942, for publication in the JOURNAL Supplement, concluding Proceedings V. 38.

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN - (Supplement) November 1942

Discussion of a paper by Gerald Pickett:

The Effect of Change in Moisture-Content of the Creep of Concrete Under a Sustained Load*

By R. W. CARLSON†

The author has made careful mathematical analyses which show that uniform and homogeneous shrinkage of concrete cannot in itself increase the plastic flow or creep of concrete. Furthermore, he shows to what extent non-uniform shrinkage across a section of a concrete member can increase the creep under a reasonable assumption of non-linear stress-strain relationship. In an actual example used by the author, however, the analysis does not account for all of the increased creep.

A possible weakness of the paper is the inference that non-uniform shrinkage across a section of concrete, combined with inelasticity, is the full and only explanation for the increased flow under drying conditions. It seems more accurate to picture at least four differences between drying and non-drying concrete which might account for the increased flow of drying concrete. The other three are: (1) local stresses within the concrete, largely due to the fact that aggregate particles tend not to shrink, the result being abnormally large creep in zones of high stress and thus an increase in the average creep; (2) possible internal cracking due to the drying, such that full correction for shrinkage is not made where tests are made on columns, and effective depth of section is reduced in beams (drying from the surfaces); and (3) there may be a direct effect of stress on shrinkage (this seems least likely, in view of the relatively small magnitude of applied force as compared with molecular forces).

Although it is difficult to evaluate these other factors not treated by the author, it seems highly desirable that research along this line be continued. Among the various tests which might be made to provide the needed information are (1) comparative tests on neat cement and corresponding concrete to show the possible effect of local stresses due to aggregate, (2) load tests on specimens which have been subjected only to

*ACI JOURNAL, Feb. 1942; *Proceedings* V. 38, p. 333.
†Massachusetts Institute of Technology, Cambridge, Mass.

free shrinkage to determine the amount of extra deformation under fairly quick loading, and (3) comparative tests on specimens which have been dried gradually to equilibrium by successive reduction in drying humidity. A few tests along the indicated lines have been made, but in general, insufficient controls have been provided.

As an example of the type of work which can be done to reveal more clearly the effect of drying on concrete behavior, a modest series of tests now in progress in the laboratories of the Massachusetts Institute of Technology might be mentioned. Beams of neat cement and concrete are being dried slowly to varying degrees, all specimens being given a preliminary moist curing period of three months. One group is then dried at 75 per cent relative humidity for a specified period, then sealed and allowed to come to equilibrium. Another group is dried at 75 per cent for the same period, then at 50 per cent for a like period, then sealed and allowed to come to equilibrium. A third group is dried successively at 75 per cent, 50 per cent, and 25 per cent humidity before being sealed. A fourth group is carried a step farther to be dried finally in an oven, for the extreme case. The relative properties of the neat cements and concretes are to be studied when the specimens of varying degrees of dryness are six months old. If the age of six months proves insufficient to eliminate the non-uniformities within each specimen, additional tests will be planned with the preliminary tests as a guide.

The author's analyses, however excellent, and such tests as are now in progress, mark only a beginning attack on the subject of the extraneous effects of drying of concrete. This seems to be one of the most fertile fields for concrete research. Since partial drying of concrete occurs so generally, its effects should be better understood.

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Job Problems and Practice

WHAT JPP MEANS

—an informal clearing house for mutual help on concrete problems; an opportunity for the ACI member who *knows* to tell the ACI member who *needs* the information. If the subject matter holds probabilities of general interest it will be briefed in these pages—with authorship credit unless the contributor prefers not.

If you know of an interesting problem whose solution has smoothed the way of someone in the field, record it briefly. If you have a problem, present it; a question, ask it. If you are moved to helpful comment or constructive criticism write the Secretary.

The "answers" are those of individuals to whom the questions are referred and not of the ACI.

Bond of Flat Steel Reinforcing (38-86)

Q—I have been trying to find the allowable unit bond stress of flat steel, say 2 in. wide, gauge No. 14, embedded in concrete; also the allowable pressure of steel on concrete of a $\frac{3}{8}$-in. edge of a $\frac{3}{8}$ in. $\times$ $1\frac{1}{4}$ in. flat steel when completely surrounded by concrete. Do you know of any experiments made on such materials and where the results of such experiments have been recorded and published?

BY HERBERT J. GILKEY*

A—The data on the bond that can be developed by bars which are flat, or by bars of other non-conventional shape of cross section are meagre, but such evidence as we have indicates that it would be reasonable to assume that for a given type of bar surface the bars should develop essentially the same bond resistance per unit of superficial area as would be developed by a plain round or plain square bar. There are, however, a few qualifying factors to which special consideration may well be given. If the bars are in the flat position as cast, the bond on the under side of them will be poor due to the film of moisture which gathers on the under side of all horizontal surfaces. If they are on edge

*Head T and A M Dept., Iowa State College, Ames, Iowa.

as cast, both of the flat sides should develop excellent bond because what little water-gain effect there is, is against a narrow face instead of a wide one. If the length of the bar is vertical as cast, as in a vertically cast pull-out specimen, the bond developed may be expected to be from 25 to 40 per cent higher than if the bars are horizontal and in the flat position as cast. The higher bond for bars vertical as cast, is due primarily to the more favorable orientation as regards the water-gain effect.

If the bars being used are at all oily or greasy or if they are smoother than an ordinary plain reinforcing rod, the bond developed by them will be substantially reduced.

Recently there has been released Bulletin 147 of the Iowa Engineering Experiment Station on the subject of bond and it might be helpful in such problems.

Regarding the allowable pressure of a $\frac{3}{8}$-in. edge against concrete when completely embedded, I have no recommendation to make nor do I know of any tests which would shed useful light on this problem.

Concreting in the Rain (38-87)

Q—What is good practice about concreting in the rain? My own notion is that in placing concrete—mass concrete—work may continue through light rain, and that concreting should be stopped when rain is heavy enough to wash out the cement. Our summer "down under" on the other side of the globe*, has frequent short, heavy thunder storms and I believe that concreting can be continued after these if the placed concrete is protected with tarpaulins, etc., during the storm. Do any specifications prohibit concrete placing in wet weather?

BY R. F. BLANKS†

A—This question brings up again a question that has been asked often recently due to the unusually wet winter on the Central Valley Project in California.

We are largely in agreement with your correspondent's idea that mass concrete work may continue through light rain and that it probably should be stopped when the rain is heavy enough to visibly wash out the cement. Although we rarely have protected fresh mass concrete with tarpaulins till the rain is over, we do take special precautions when placing is resumed to see that there will be no plane of weakness. Depending on the time elapsed and the amount of cement lost, these precautions may range from extra vibration to weld the new concrete to the old, through use of a mortar batch at the start, to a complete cleanup and mortar batch as on an ordinary construction joint.

*The question comes from Queensland, Australia.
†Bureau of Reclamation, Denver, Colo.

The general statement which follows may be found in the Bureau of Reclamation Concrete Manual, Third Edition, page 293:

The engineer in charge of concrete placement is at times confronted with the problem of deciding whether concreting may be continued during a rain, and if continued, what precautions are necessary. As a quick decision must be made, trouble may be forestalled by thoughtful pre-planning based on job conditions and current weather reports. A reasonable supply of protective covering should be available for immediate use. If periodic or sustained rainfall is imminent, the scale and arrangement of operations should be so devised as to facilitate protection of the work. Owing to the large variety of job conditions, it is impracticable to include in this manual any comprehensive definite procedure for concreting in rainy weather. It may be said, however, that the work must be discontinued when the rain is so severe that water collects in pools or washes the surface of the fresh concrete and it is infeasible to provide adequate shelter.

Specifications do not cover this point directly but do contain the requirement that no concrete shall be placed in water without special written permission and then only if an approved method is used. This is invoked when puddles accumulate on the concrete in the stage between a light rain and one heavy enough to stop the work. The approved method is to sponge up this free water before depositing new concrete. If rain continues at this severity, this procedure becomes so costly that placing is discontinued.

The most critical stage in placing concrete during even light rain is the spreading and protection of the mortar layer on construction joints between lifts. Thicker layers, stiffer consistencies, spreading small areas at one time and temporary coverings are expedients which may be employed to a limited extent.

At the minus 2-inch slump consistency that it is practicable to place mass concrete by means of vibration equipment now available, it is infeasible to make the concrete any drier to compensate for rain water. Trouble with such drier concrete results immediately in handling equipment. It is also doubtful if the rain water will become suitably and uniformly mixed in the drier concrete during placing.

The inconvenience and expense of interruptions and delays due to rains is recognized but, since control of water in the concrete is the greatest element in job control of concrete quality, it does not seem prudent to be too lenient in this case merely to avoid interruptions.

Slag Aggregate—Wet or Dry (38-88)

BY FRED HUBBARD*

Q—Should slag used as concrete aggregate be wet before use?

A—Under some conditions the pre-wetting of slag aggregate before use in concrete is undoubtedly an advantage. However, by far the greatest tonnage of slag concrete aggregate is used dry and, in some instances, this also would be a distinct advantage. If slag is used as concrete

*Director of Research, National Slag Assn., Youngstown, Ohio.

aggregate in a comparatively dry mix, and in such construction as a pavement slab, pre-wetting the slag, especially during hot dry summer months and on windy days, is advantageous in that it prevents any tendency of the dry slag to take water from the mix after the concrete has been placed and cause the concrete to dry out too rapidly for proper finishing.

On the other hand, if slag is used in a comparatively wet mix, especially in cold damp weather, the use of the material dry is undoubtedly an advantage, allowing the aggregate to absorb some of the excess mixing water and produce a better quality of concrete. Many slag producers wet slag which is to be used as concrete aggregate at the slag plant after it is crushed and screened. In a few instances a screening process is accompanied by washing, in which case, of course, the slag is delivered wet.

A direct answer to this question is, therefore, that slag is successfully used as concrete aggregate in both wet and dry condition, and that the pre-wetting of the slag is more or less a matter of opinion and any advantage derived from wetting the material will depend largely on field conditions.

What Concrete Coverage to Protect Reinforcing (38-89)

Q—What information is there on the life of reinforcing steel with ¼ to 1 in. cover, in a concrete slab exposed to moist air approaching the saturation point at a temperature of 100 to 110 F.

The slabs in which these rods are enbedded are exposed to frequent periods of drying out, i. e. alternate wetting and drying of the slabs. This condition is certainly more conducive to corrosive action in the reinforcing than one where the slab is wet continuously.

I would also like to know if any tests have been made, or if there are records of any kind, of the use of reinforcing rods previously coated with a cement, i. e. cement and water. If this is a good precaution to prevent reinforcing rods oxidizing when the cover is very thin—and if it has been used for this purpose, somebody must have developed a proper technique for applying it to the reinforcing steel. Is there any information about how such a coating may be applied on a commercial scale?

A—We know of no tests where various amounts of coverage were used over reinforcing rods and the concrete exposed to moisture conditions as described. Offhand the only example which we can cite of reinforcing steel with very little covering used under conditions likely to promote rusting are ships or structures exposed to sea water.

During the last war a scow was built in Seattle and a concrete water barge was built at Vancouver which had little coverage over the steel.

The covering over the steel was well under ½ in. in many places. Both of these vessels have had pretty continuous service around Puget Sound. Examined in 1939 there was practically no evidence of rust or corrosion of the reinforcing, although the Seattle built scow had numerous fine shrinkage cracks visible in her sides. Practically all the rust spots noted were around bolts, spikes, etc., which were used to attach fender timbers to the concrete hull. The Vancouver barge did not have a single rust spot on it although at one place on the edge of her deck the tie lines had worn away the concrete and exposed the reinforcing so that one bar had a ⅛ to ¼-in. uncovered spot but no rust had occurred. However, it must be kept in mind that these barges were made with rich concrete, that is, high cement content and low water-cement ratio.

In 1929 an examination was made of the Million Dollar Pier at Miami which had been seriously damaged by the hurricane of 1926. In examining broken sections from which bars had been protruding for nearly 3 years, it was rather reassuring to find, when the concrete was broken away, that the steel was protected right out to the outer layer of contact. The significance of this observation will be more appreciated when it is remembered that in the sea coast region corrosion of all steel exposed to the salt spray, as was the case with this structure, is very rapid.

It is our opinion from these examples and other experiences that if the concrete placing is such as to positively prevent cavities from forming under the bars, 1-in. protection of good concrete would be ample and corrosion would not progress lengthwise along the bars. Studies of concrete mixtures, however, have indicated that cavities can form readily not only under the bars but under some of the larger stone particles in the mixture. Cavities form under reinforcing bars, particularly under the upper bars in deep beams where wet concrete is being used. The bars being held rigidly in place, it is to be expected that the concrete would settle away, leaving the cavity. It is more noticeable under the upper bars because of the greater depth of concrete below the bar. However, with all but very stiff mixes the same tendency exists with bars held close to the lower surface. and it needs only a slight cavity for the rust to progress.

Practical Specifications for Structural Concrete (38-90)

Q—My problem is that of writing a specification that will be effective under our method of procedure. In our work with structural concrete, it seems insufficient to say in the specifications that the slump of concrete shall not exceed 4 in., that chutes, hoppers, etc., shall be of pitch and design to permit the expedient handling of concrete within this limit of slump. In some cases, it is not feasible to have rigid and continuous

inspection. If an inspector is not on the job, foremen allow chutes to become too flat or concrete is no longer assisted through the bottlenecks and then the claim is made that they just couldn't handle concrete any stiffer than a 6- or 7-in. slump, although concrete within the specified slump would work readily in the forms.

After concreting starts and faults in the equipment become apparent, it is usually too late to change; it seems necessary to make the specifications so detailed and so definite that there will be no doubt in anyone's mind as to just what the contractor will be required to provide in the way of equipment. Can you refer me to manufacturers of suitable equipment or to suitable published specifications of this type?—H. G. Collins.

BY LEWIS H. TUTHILL*

A—Your difficulty is not unique, and your idea of more detailed and definite specifications is part of the answer. However, I believe it is not possible to write a specification that will insure results without competent, full-time inspection. In other words, I have long ago come to the realization that it is a literary impossibility to write a self-administering set of specifications.

There are two possibilities in writing specifications. One is to describe the results which are to be obtained and let it go at that. The other is to describe the results and specify the use of certain well-demonstrated, successful methods by which the results can be obtained, and prohibit methods which are known to be unreliable and often productive of unsatisfactory results. You are certainly right that a more complete type of specification is necessary. Too often under the "results only" type of specification, the contractor moves in with a setup that cannot possibly produce results that are satisfactory; yet nothing can be done until the plant is in operation and the results again prove it to be incapable of attaining specification requirements. By that time it is usually too late to alter the plant, for the work must go on.

As source material for the specific, detailed type of specification, such things as the report of ACI Committee 614† and the figures it contains, have been prepared. There are many other sources among ACI publications and the various concrete manuals such as those of the ACI, Santa Fe Railroad, and Bureau of Reclamation. Experience is another invaluable source of ideas that should influence each new specification.

I know of no specifications to which I can refer you that cover your particular work. General specifications are worthless. A specification must be written expressly for a job, taking into consideration all the special aspects of that particular job. A clearly written specification,

*Chairman ACI Committee 614, Measuring, Mixing and Placing Concrete, Denver.
†ACI JOURNAL Nov. 1941; *Proceedings* V. 38.

stating exactly what is expected, and how it is to be done, and what is to be prohibited, will withstand the test of administration on the work and will result in fewer lawyer's fees than will the more obscure general type of specification.

A common equipment setup uses a conveniently located hoist tower to supply concrete to a floor hopper from which buggies carry the concrete to the forms. Numerous manufacturers make such equipment but you can't hold them responsible for a chute from the mixer to the skip loading hopper, for instance, that is too flat to carry concrete of the proper consistency. Nor are they responsible if the skip discharge hopper on the tower is not moved up often enough or far enough to afford adequate slope of chute to the floor hopper. These things are purely matters of operation (among others) that your specifications should provide for and that your engineering and inspection forces see that you get.

The third element of a military operation, long ago cryptically defined, is equally important in construction operations. The first two elements are: (1) have a plan; (2) have the force to carry it out. The all important third: See that it is done!

The Shrinkage of Concrete (38-91)

Q—Most existing information on the shrinkage of concrete pertains to small laboratory specimens dried artificially. These specimens presumably not only are smaller, but are dried more completely. How can results of these laboratory tests be used to predict the probable shrinkage of concrete in such structures as buildings?

BY ROY W. CARLSON*

A—This question is partially answered by a paper in the ACI JOURNAL Jan.-Feb. 1937, entitled "Drying Shrinkage of Large Concrete Members". In that paper, the effects of size of section and time of drying on degree of drying and shrinkage are shown. It is shown, for example, that a 6-in. slab may lose 20 per cent of its evaporable water in one month, while several years are required for a 48-in. slab to lose 20 per cent.

The important information still lacking is the maximum amount of drying and shrinkage which occurs in buildings, where the drying is not continuous. A building wall may absorb as much water in one rain as it loses in several weeks of drying. Therefore, it is expected that the drying will not proceed indefinitely in a service structure, but will reach some equilibrium condition, where the absorption during wet seasons offsets the drying of other seasons. The final result will undoubtedly depend upon the climatic condition and upon the exposure of the particular concrete.

*Mass. Institute of Technology, Cambrdge, Mass.

Why the Limitation on Extreme Fiber Stress (38-92)

Q—Why is design extreme fiber stress for concrete in compression limited to 0.45 of the 28-day strength when test specimens of balanced design, so based, invariably fail in tension?

BY F. E. RICHART*

A—This working stress is presumably based primarily on the tests made by Slater and Zipprodt, *Proceedings*, ACI, 1920, p. 120, and by Slater and Lyse, *Proceedings*, ACI, 1930, p. 831. Another paper bearing on the subject is by C. B. McCullough, "Flexural Resistance of Shallow Concrete Beams", *Eng. News-Record*, Sept. 19, 1935. Slater and Lyse showed very definitely that in beams which actually failed in flexural compression, the maximum compressive fiber stress (on the basis of the straight-line theory and a value of n as given by the ACI Code) was more than 1.5 times the compressive strength of control cylinders of the same concrete, for concrete of strength 4000 psi and less. For concrete of strength greater than 4000 psi, the ratio varied from 1.5 to 1.45. Comparing the smallest of these ultimate strengths, $1.45\,f'_c$ with the design stress $0.45\,f'_c$, the factor of safety is found to be 3.2. A similar comparison of a working stress of tensile steel of 20,000 psi with the specified yield point value, for intermediate grade steel, of 40,000 psi, with allowance for a slight reduction in moment arm at the higher stress, shows that the factor of safety for the steel may be 2.0 or less. This should show pretty clearly why beams designed with these stresses fail in tension.

The reason for using so-called balanced "reinforcement" when we know that, with the working stresses used, it is not actually balanced, apparently is based upon a doubt on the part of engineers that adequate inspection will be provided for reinforced concrete construction. It is probably felt that a lower factor of safety may be permitted with steel, a factory made product (even though it is not inspected), than should be permitted with the concrete, which may be subject to many ills of proportioning, mixing, placing and curing. At least such a tendency has been apparent from the working stresses used for reinforced concrete from the beginning of its use in this country.

This conservatism with regard to flexural stresses in reinforced concrete may be justified by the fact that the use of higher concrete stresses would produce but little saving in cost, since reducing the depth of flexural members would increase the amount of reinforcing steel required. The principal economy then would be in saving of dead load.

*Research Professor of Engineering Materials, University of Illinois, Urbana.

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Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Minimun-steel graphs for bending and compression

P. Massarik. *Concrete and Constructional Engineering*, V. 36, No. 9, p. 353-364, Sept. 1941.
Reviewed by Glenn Murphy

The author presents charts to facilitate determining the most economical areas of tension and compression steel for any rectangular section under bending and compression (tension being developed) or for any rectangular section in simple bending where compression steel in required. The method outlined also applies when the cover ratios are different on the two faces of the section. Several illustrative examples are included.

Proportioning of concrete

K. E. Andrews, B. E., *Journal of the Institution of Engineers Australia*, V. 13, No. 8, Aug. 1941, p. 181.
Reviewed by J. R. Shank

The author summarizes very well the history of the theory of proportioning concrete through to the present day. The references are generally American. The presentation is thorough, succinct and interesting. This paper might well appear in any good text on concrete. No abstract is made because all of the material can be found in any good library and much of it is in the *Proceedings* of the American Concrete Institute.

Lining the world's longest tunnel

Engineering News-Record, V. 127, No. 23 (Dec. 4, 1941), p. 70-75. Reviewed by S. J. Chamberlin

Concrete lining is under way in the Delaware water-supply tunnel with a large array of equipment and divergence of schemes. One contractor mixes concrete at the surface where it discharges through a 10-inch pipe to 375 ft. below the surface for a long haul in special agitator cars. Another batches at the surface, mixes with double-drum paver in the tunnel and places by pipe-line. Another delivers concrete at the surface by truck-mixers which drop the concrete through 10-in. pipes.

Designing reinforced-concrete beams with variable moments of inertia

Charles A. Ellis, *Engineering News-Record*, V. 127, No. 19 (Nov. 6, 1941) p. 84-87.
Reviewed by S. J. Chamberlin

The method of determining angles of rotation and moments of fixed end-beams by the principle of area moments for beams of constant moment of inertia is extended to beams of variable moments of inertia by dividing the beam into a number of units and applying area moments to each. End moments are obtained by summing the values of the units. The procedure is illustrated for several types of loading. A table of key coefficients of universal applicability is included.

Cantilever forms preferred at Mahoning

KENNETH C. COX, *Engineering News-Record*, V. 127, No. 17 (Oct. 23, 1941) p. 86-88.
Reviewed by S. J. CHAMBERLIN

Designed for 5-ft. lifts, rolled steel sections were used for the horizontal and vertical wales of the forms with 4-in. tongue and groove sheathing. Rods and stud bolts were of high tensile steel. The bottom stud bolt accommodated two nuts and washers for adjusting the top of the form to any position. At the top of the vertical wale a cantilever steel leg supported the form on the hardened concrete while the ties were being removed.

Precast facing used on Navy hospital

HUGO C. FISCHER, *Engineering News-Record*, V. 127, No. 17 (Oct. 23, 1941) p. 108-111.
Reviewed by S. J. CHAMBERLIN

Some 400,000 sq. ft. of precast exposed aggregate concrete panels were used to face the 3- and 4-story concrete frames and 20-story structural steel tower. Five-hundred different shapes of panels were required. Integral lugs furnished a positive tie to the brick backing. The panels are free-standing, being anchored to but not supported by the concrete frame. The use of quartz aggregates and white cement created a distinctive texture.*

Artificial stone cornices

Beton-Stein Zeitung, Vol. 14, No. 10, May 25, 1941. Reviewed by A. U. THEUER

The use of concrete cornices even where all other architectural details in a building may be limestone is stated to result from a certain tendency in modern architecture favoring a large overhang. This is attributed to difficulties with natural stone both in securing suitably large pieces of stone and providing for proper anchorage into the wall. Practical methods for pouring the cornice on the job, both in situ or on the ground, are described.

Navy builds concrete hangars at San Diego

Engineering News-Record, V. 127, No. 23 (Dec. 4, 1941), p. 64-66. Reviewed by S. J. CHAMBERLIN

The structures are of the thin-shell barrel-type, each consisting of four independent sections separated by expansion joints. Two-hinged arch ribs are supported on piles and tied together by bridge strands. The movable arch centering and forms for the main roof were designed so that one complete roof unit could be placed monolithically. The concrete was brought to the top of the hangars by pipeline and distributed by metal elephant trunks and auxiliary chutes.

Drydock concreting by belt conveyor

Engineering News-Record, V. 127, No. 21 (Nov. 20, 1941), p. 84-86. Reviewed by S. J. CHAMBERLIN

A system of 24-in. belts conveys the concrete at rates up to 180 cu. yd. per hr. The first belt receives from the hopper under the mixer at the center of one side of the 135 by 720-ft. area and delivers it to a belt on top of a trestle extending along one side. The belt on the trestle can move in either direction delivering at any point along the full length. A traveling steel bridge spans the area and carries two belts, one from the trestle belt to the center, and a second belt which is reversible so that it can deliver to any point under the full length of the bridge. A movable hopper and delivery pipe are suspended beneath the bridge. Uniform loading and cleaning of belts is accomplished effectively.

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*See p. 289 this JOURNAL.

Steam-curing in concrete and lightweight-concrete products

Beton-Stein Zeitung, V. 14, No. 10, May 25, 1941. Reviewed by A. U. THEUER

General test data are presented, indicating the loss in weight and shrinkage against time of three different cement-aggregate proportions for three methods of curing. The three methods of curing are (1) steam pressure at 180 C., (2) damp curing at room temperature, and (3) curing by subjecting specimens to a combined steam and flue-gas treatment. Data are also presented to indicate the increase of strength with time for three steam curing temperatures (100, 140, and 180 C.) following initial curing periods of 2 and 6 days at room temperature. A number of conclusions with regard to steam curing are drawn from general practice.

Cement dispersion and concrete quality

E. W. SCRIPTURE, *Engineering News-Record*, V. 127, No. 23 (Dec. 4, 1941) p. 82-85.
Reviewed by S. J. CHAMBERLIN

Dispersing agents are used to spread the individual cement particles apart, that is to avoid flocculation. Water that would be trapped in cement clumps is released to become a part of the placing water, thereby making it possible to reduce the water-cement ratio. The surface area of the cement in contact with water is greatly increased which increases the rate of hydration. Unlike admixtures which attempt to add some extraneous material for beneficial effects, dispersing agents merely increase the efficiency of the cement itself. Mixes of the same design and consistency show increased resistance to freezing and thawing and increased strengths when dispersing agents are used. The cost is roughly equivalent to that of three-quarters of a sack of cement per cubic yard of concrete.

War-time buildings—three-pin portal frames

Concrete and Constructional Engineering, V. 36, No. 9, p. 365-374, Sept. 1941.
Reviewed by GLENN MURPHY

Restrictions upon steel and timber in England together with the necessity for fire resistant construction which will comply with special loading conditions and in addition be easily camouflaged have led to the development of precast reinforced-concrete frames. The system, evolved by Mr. C. W. Glover, utilizes standardized precast reinforced-concrete shapes for spans up to 70 ft. The frames are cast in two halves and, in erection, are pinned at the base and at the crown where the two halves meet. The members are designed to facilitate the placement of purlins and other appurtenances. The three-pin construction, being statically determinate, adapts itself to large movements of individual foundations due to uneven settlement or displacement due to bombing effects. The three-pin portal frame is also adapted to "shadowless" construction.

Sulfate resistance of portland cements

Technical News Bulletin of the National Bureau of Standards, No. 292, Aug., 1941, p. 70.
HIGHWAY RESEARCH ABSTRACTS

In extensive areas of the West-Central portion of the United States and Canada, many clays and soils occur which contain several per cent of sulfates, particularly sodium sulfate. The action of these "alkali" soils on portland cement mortar or concrete results in the conversion of the hydrated calcium aluminate of the set cement to calcium sulfoaluminate and gypsum, accompanied by marked expansion. Complete disintegration of the mortar or concrete may eventually occur.

Methods by which the sulfate resistance of portland cement mortar or concrete may be improved, include the following: (1) Steaming the mortar or concrete; (2) prolonged curing of the mortar or concrete at ordinary temperatures; (3) mixing the cement with

a puzzolanic material; (4) increasing the glass content of the cement; (5) altering the chemical composition of the cement.

Experiments by E. P. Flint and Lansing S. Wells, which are reported in the *Journal of Research* for August (RP-1411), show that the above methods are effective because they bring about the removal from the set cement of tricalcium aluminate hexahydrate, which is unstable in sodium sulfate solutions, and the formation in its place of silica- or iron-containing hydrogarnets, which are stable in such solutions.

Products having an X-ray structure nearly identical with that of grossularite garnet were obtained when mixtures of beta-dicalcium silicate, lime, and dehydrated kaolin were steamed at 500 C. and 420 atmospheres. Dicalcium silicate and tricalcium silicate are major constituents of portland cement and dehydrated clay, or kaolin, is a common puzzolanic cement admixture.

The effect of deflection of the supporting beams on the bending moments in a uniformly loaded slab

ALAN BURNS. *The Journal of the Institution of Engineers, Australia*, Vol. 13, No. 6 of June 1941, p. 131.
Reviewed by J. R. SHANK

The author makes a thorough mathematical analysis of the problem of determining the bending moments for a ribbed slab freely supported. The slab is fully continuous over the rib beams. The slab and rib beams are freely supported on the walls. Two types of ribs are considered, steel I-beams and concrete T-beams.

Mathematical expressions are developed and constants are worked out and presented in tabular form. These tables are also expressed in the form of curves. Two problems, one with I-beam ribs and one with T-beam ribs are worked out in detail. These results are compared with similarly calculated results for the case in which the deflections of the rib beams are assumed as zero.

The bending moment for the ribs are only slightly less than those calculated for simple beam action for tributary loads for mid-span to mid-span of slab. The moments for the complete analysis taking into account the deflection of the rib beams, is the smallest of the three, about 80 to 85 per cent of the largest or rough simple beam values.

The bending moment in the slab for the short way continuous slab action over the rib-beams was very nearly the same for ordinary continuous beam action and for the more accurate analysis considering no rib beam deflection, but the bending moments from the analysis considering the deflections of the rib-beams was 60 per cent of that for ordinary continuous beam action for the steel I-beam rib construction and 75 per cent for the concrete T-beam ribs. The situation of most interest is the bending moments obtained for the slab bending parallel to the rib beams. The bending moments in the I-beam rib construction at mid-panel point and over the mid-span point of the rib-beams were nearly twice as great as any bending moment for the short span over the rib beams.

The general observation from a design standpoint is that ordinary two-way supported slab computations are not correct for the long-way bending, that the deflection of the rib beams causes large bending in the slab. The author suggests that the main slab reinforcement be placed parallel with the rib beams. He says nothing about what might happen if no reinforcement other than temperature reinforcement were placed parallel with the rib beam or that a few cracks ar right angles to the rib beams might not cause any particular trouble.

Types of powdered stone in concrete road construction

WEBER, *Betonstrasse*, 1940, 15 (1), 5-7 Road Abstracts. 1941, Vol. VIII, No. 5, p. 55, London. HIGHWAY RESEARCH ABSTRACTS

German official specifications for the construction of concrete surfacings permit the addition of powdered stone in a proportion not exceeding 2 per cent of the total weight of the aggregates. Since the minimum proportion of fines in the mix is fixed at 3 per cent, the aggregates should themselves contain at least 1 per cent. The fact that this proportion is seldom present in river sand or washed sand calls for some modification in the specification requirements. Other German specifications demand from 3 per cent to 8 per cent of fines. Reference is made to the view taken by Hummel that the optimum proportion of fines depends on the type of cement. The author compares the properties of seven types of powdered stone in use in Silesia. The cements used in concrete surfacings in this area are described as "standard", as distinct from "road" cements, and the proportion of fines usually employed is 4 to 5 per cent. It has been found that higher and lower proportions result respectively in an undesirable amount of laitance and in the separation of water at the surface. Locally available types of powdered stone include trass, powdered basalt, Oderwalde sand, two types of hydraulith (a local basaltic tufa) powdered limestone, and Thurament (a product similar to trass). A series of comparative tests was carried out on concrete containing 617 lb. of cement per cu. yd. with mixes containing 0, 3, 5 or 7 per cent of powdered stone. The properties tested included the compressive strength of cubes at 7 and 28 days, the bending strength of specimens 40 in. by 4 in. by 4 in. at 7, 28 and 90 days, and of specimens 28 in. by 6 in. by 4 in. at 28 days, and the shrinkage at 7 and 28 day, 3 months, and 6 months. The following conclusions were reached: (1) For the cement content employed the optimum proportion of added filler is usually above the official specified maximum of 3 per cent; the concrete is usually improved by increasing the proportion of fines up to a total of 5 per cent. (2) The amount of shrinkage was greatest when powdered limestone or hydraulith was used, and was least with Thurament and trass. (3) The highest bending strengths were observed when the proportion of filler did not exceed 5 per cent, though the filler content of the shorter beams could in some cases be increased to 7 per cent without causing an appreciable reduction in strength. (4) The compressive strength was lowest when limestone or sand was used, and highest with Thurament, trass and powdered basalt.

Materials testing

H. J. GILKEY, GLENN MURPHY and E. O. BERGMAN, 1st Ed. 185 pages illustrated. McGraw-Hill Book Co., Inc., $2.75. AUTHORS' REVIEW (by ACI request)

Covers theory, practice and significance of physical tests on engineering materials. Contains 12 chapters of text covering testing, testing equipment, and testing observations; properties of materials; and chapters on specific kinds of tests such as tensile, compressive, shearing, flexural, column, hardness, impact, and fatigue. Also covers use of strain gages and has an important chapter devoted to the design, control and curing of concrete mixtures and another on experimental aids in stress analysis. There are 19 formal problems or experiments and 325 questions, largely of the "straw-man" type, designed to focus attention upon specific aspects, extensions and applications of materials testing. All such questions are answered and discussed, the actual coverage or scope of the book being greatly extended thereby.

Throughout a close linkage is established with the work and publications of the ASTM and with the literature of such other organizations as the ACI. The scope exceeds that likely to be desired by any particular school but the book is intended to be inclusive in the sense that any institution may find therein all that it cares to use, or

can use within the limitations of the time and equipment available. The Preface and the Foreword to Instructors supply specific suggestions for selections and omissions to meet different situations.

The chapter on concrete is original in concept and execution. Moreover, although highly condensed, it endeavors especially to introduce, define and illustrate the terms and the varied language and units of concrete for both the laboratory and the field. Thus one will find defined and numerically illustrated such terms and items as yield, voids, voids-cement ratio, water-cement ratio, cement-water ratio, solidity ratio, net water content, bulking, fineness modulus and surface modulus. Trial batch design, Lyse's method for adjusting designs, expression of quantities by absolute volume, loose volume and by weight and corrections for absorption, free moisture, bulking and air voids are all covered and illustrated as are the effects of grading and maximum size of aggregate upon workability and economy of mixtures. Several experiments designed to illustrate both the adaptations and the limitations of current design methods and practice, are outlined in detail. The treatment also includes mention of and definite references to such important recent questions as: tallows and resins in relationship to air voids and resistance to frost action; alkalies in cement as a possible source of disasterous interaction between the cement and certain coarse aggregates in concrete; and the latest ASTM specifications establishing as standard several different special-purpose portland cements. Additions and admixtures are mentioned and distinguished one from the other.

The chapter on experimental aids in stress analysis covers much material not previously available in a course of this kind.

The book is well documented and thoroughly indexed so that any topic touched upon anywhere in the book can be located readily and any topic mentioned can be pursued further with a minimum of effort. The large page size, 8½x11 in. and the "open-flat" binding were selected to facilitate use in the laboratory where it is desirable that instructions be found and followed with little turning of pages.

The book contains much material well suited to graduate work and to self study. It is also well adapted for use in defense training courses for men who have an elementary background in mechanics.

First Annual

TECHNICAL PROGRESS SECTION

FOREWORD

In the discovery, evolution, and application of the progressive knowledge of the construction field, there are two cooperating groups of engineer-scientists. They are commonly regarded as distinct in their viewpoints. In theory, one of them has the professional viewpoint with its desire to serve the consumer directly or indirectly, while the other group represents commercial enterprise for the technical development of special materials, equipment, and services, tactily accepting a responsibility to make its products serve the best engineering principles. Working from its special viewpoint each succeeds as it serves the same master—the public interest.

Each group has something to sell to the other and members of both are often conservative and hesitant—about adopting new ideas in one case or new products in the other. Insofar as these two groups are identified with the field of concrete, ACI provides opportunity for viewpoints to meet in mutual respect and understanding. Neither succeeds without the other.

In a man's own specialized field he is usually scornful of ballyhoo; he is alienated by overstatement, by breezy generalities, by some of the highly imaginative commercial promotion of panaceas. The engineer-buyer, nurtured in the way of factual precision, is critically minded; he asks searching questions, looks for well supported facts, is cold to claims so sweeping that they insult his informed intelligence. But, once sold, he becomes a major influence in the adoption of a

better product or a better way. He holds the key to many a sale. If the product or service is good, there should be no delay about using it.

In these broad statements is the basis of an Institute departure in commercial advertising policy. In them is the inception of the First Annual Technical Progress Section of the ACI JOURNAL.

In the 48 pages of this Section of the JOURNAL, following a list of the participants, are their messages—messages from leaders in developing, promoting and producing materials, equipment, and services in the field of concrete. They use this opportunity to say their say to the Institute's specialized audience.

Institute endorsement is neither given nor implied. It has been the purpose of the Institute's Publications Committee to encourage in these advertising pages (insofar as its suggestions could be made effective) that type of well supported factual statement which might contribute to the prompt acceptance of methods, materials, and equipment which have something to add to the sum of progress in the Institute's field. Advertisers have caught the spirit of the occasion, the significance of the special opportunity, in various ways—often with that commendable sense of restraint which should impress intelligently critical ACI readers.

The Publications Committee presents these pages to the Institute's audience for consideration. Of their significance that audience will be the judge. Its judgment may help to show how to improve such an opportunity as this, next time, in the Second Annual Technical Progress Section of the ACI JOURNAL, February 1943.

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Farmers Bank Bldg. • Pittsburgh, Pa.—New York, Chicago, Birmingham, Philadelphia

TRUKMIXER LOADING PLANTS

Blaw - Knox Trukmixer Loading Plants include complete equipment for overhead storage of aggregates and bulk cement; also accurate weighing batchers (either manual or automatic) for measurement of aggregates, bulk cement, water, and admixtures.

Many of these plants are made up of an assembly of standard bin and batching units. Blaw-Knox also designs special plants to fit special requirements and conditions. Accuracy of proportioning; speed, economy, and ease of operation, sturdiness and durability of equipment are stressed in the design of Blaw-Knox Trukmixer Loading Plants.

BULK CEMENT PLANTS

Complete Blaw-Knox Bulk Cement Plants include equipment for unloading cement from railroad cars or trucks, elevators for elevating cement into storage bins, the overhead storage bin, and weighing batcher equipment for measurement of cement.

They are available in a portable design which provides for easy and economical erection, dismantling, and moving. Blaw-Knox also furnishes larger bulk cement plants where large overhead storage is required, or for several brands of cement.

The weighing batcher equipment is either of the manual or automatic type, designed for easy operation and extreme accuracy. The storage bin equipment is sealed for protection of the cement against weather.

ROAD FINISHING MACHINES

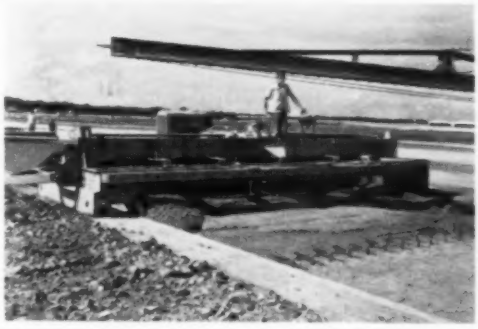
The Blaw-Knox two-screed finisher for concrete paving for roads and airports is arranged for utmost flexibility in operation to take care of varying textures of concrete. The machine is equipped with six traction speeds, forward and reverse, and four screed speeds. Screeds are adjustable for crown; also, screeds with quick crown change device for flattening crown on curves for road paving are available.

Vibratory attachments for concrete paving are also available with the Blaw-Knox Finishing Machine. Accessories are available for use of the Blaw-Knox Finisher on high type asphalt paving construction.

• BLAW-KNOX •

CONCRETE SPREADERS

The Blaw-Knox paving Spreader has had a wide application to concrete paving on roads and airports. It is automatic in its action regardless of where the concrete is placed between the side forms. It spreads concrete for the base course, ready for the placing of the mesh reinforcing and then spreads the upper course of the concrete over the mesh to finish grade ready for the finishing machine. The spreading blade and final strikeoff are quickly and easily adjustable for elevation. The Blaw-Knox Concrete Paving Spreader easily maintains pace with the largest paving mixers in use.



SPREADER-VIBRATORS

This machine is a combination of the Blaw-Knox Concrete Paving Spreader and a vibratory attachment. Spreading the concrete for the slab and vibrating it take place simultaneously. The machine is one-man operated. It provides a uniform and thorough compaction of the concrete, thus giving a dense, sealed slab. No part of the vibrating equipment rests on the road forms, thus settlement of side forms with soft soil conditions is not aggravated by the vibration.

CONCRETE BUCKETS

The Blaw-Knox Roller Gate Concrete Bucket is advantageous for the handling of mixed concrete of any consistency. The discharge gate consists of a flexible rubber pad mounted on anti-friction rollers. This design of gate can easily be closed before discharge of the complete load from the bucket. It is tight and does not leak grout. The discharge opening at the bottom of the bucket is large, and the hopper slopes are steep insuring rapid and complete discharge of dry concrete. These buckets are available in sizes ranging from $\frac{3}{4}$ yard to 8 cubic yard. The buckets are handled either by crane or cableway.

LEHIGH EARLY STRENGTH CEMENT

★ ★ ★ **Develops service strength in one-quarter to one-half the time required for normal cement**

★ ★ ★ **Eliminates hazards incidental to long curing periods in cold weather**

★ ★ ★ **Provides an additional factor of safety for architects, engineers, and contractors**

Those in charge of construction should know the type of cement best suited for a given project under given circumstances. They should know what factors govern the design of concrete mixtures, and what combination of cement, aggregate and water gives emphasis to such properties as strength, volume change, impermeability and weather resistance.

★ ★ **LEHIGH PORTLAND**

LEHIGH CEMENT

CONTRIBUTES TO PROGRESS

★ ★ ★ **Form lumber costs cut 30 to 50 per cent**

★ ★ ★ **In cold weather, fuel and labor for maintaining artificial heat for curing cut 30 to 50 per cent**

★ ★ ★ **Construction expedited**

★ ★ ★ **More uniform, better concrete**

To provide such information the Lehigh Portland Cement Company maintains a Service Department for field work, and a Modern Research Laboratory for chemical and physical research in cement and concrete problems.

The services of these departments are available without obligation.

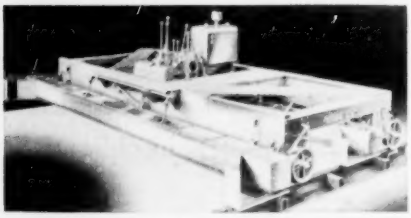
CEMENT COMPANY ★ ★

LEHIGH CEMENT

25 Years Ago, the First Finishing Machine--Today, the Spreader-Finisher Combination -- Both Are Jaeger-Lakewood Contributions To Paving!

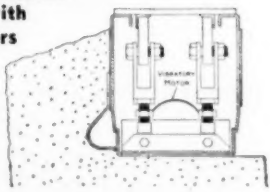
In 1917, The Lakewood Engineering Co. (now consolidated with The Jaeger Machine Co.) introduced the *first mechanical finisher* for pavements. In 1930 Jaeger-Lakewood produced the *first vibratory finisher* and followed a year later with the *first mechanical concrete spreader.* This spreader, replacing hand shovelers, was developed for use with Jaeger *side discharge truck mixers or "auto-pavers,"* another exclusive Jaeger-Lakewood contribution to paving advancement.

Jaeger-Lakewood improvements in finisher design, over 25 years, have kept pace with progress toward bigger pavers, drier mixes. In 1940, when the Pennsylvania Turnpike faced the incredible task of pouring the last 148 miles (4½ million sq. yds. of slab) in 90 days, 25 Jaeger Concrete Spreaders and 24 high speed Jaeger-Lakewood Finishers made this world's record possible. With this team, it is now possible to PAVE AS FAST AS YOU CAN POUR even where 34E dual drum pavers are used on half width work with concrete as dry as ½" slump.



MAXIMUM FINISHING CAPACITY AND FLEXIBILITY to meet today's conditions are provided by 4 gear-selected speed ranges of travel, independent gear-selected screed speeds, smooth, positive 4-wheel drive and hydraulic screed lifts. Complete transmission mounted in front where operator controls any function by single direct lever movement. Telescopic frame permits 4 ft. built-in width change. Spring cushioned, fully enclosed screeds, wider end shoes. Quick crown change, telescopic transportation unit. Write for Catalog.

The Original Successful Vibratory Method—18" Bullnose Front Screed with Vibratory Motors



Gives DEEP INTERNAL vibration to mass of concrete ahead of and under screed—not mere surface vibration. Proven by 12 years' use during which this Jaeger-Lakewood method has unfailingly met every test by Highway Depts.

Spread Any Standard Width, 9 to 25 Ft.

JAEGER SPREADER cuts cost and eliminates bottleneck of hand shoveling, enables dual drum paver and finisher to work at capacity . . . Only Spreader that guarantees against segregation (screw positively remixes, densifies and compacts). Strikes off for both mesh and top course. Write for Catalog.

FORM TAMPERS (do 6-8 man job in ½ day), EXTRA GAUGE ROAD FORM (to support today's equipment), BITUMINOUS PAVERS, BITUMINOUS SPREADER-FINISHERS, ADJUSTABLE STONE SPREADERS, MIX-IN-PLACE ROAD BUILDERS, MOBILE AUTO-PAVERS, CONCRETE AND BITUMINOUS MIXERS, SURE-PRIME PUMPS, CONTRACTORS' HOISTS. COMPLETE CATALOGS SENT ON REQUEST.

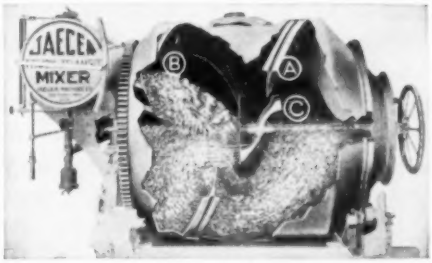
Jaeger Research Has Improved the Quality of OVER 50,000,000 CU. YDS. of TRUCK-MIXED CONCRETE

MORE THAN 1400 JAEGER TRUCK MIXERS DELIVERED FOR DEFENSE WORK IN THE PAST 12 MONTHS Emphasize the Economic Importance of this Mobile, Flexible Type of Mixing and Placing Plant and the Importance to You of 2 Factors Affecting the Quality of the Product

(1) POSITIVE REVERSING ACTION, with THROW-BACK Blades, Proven Essential to Fast, Thoro, End-to-End Mixing

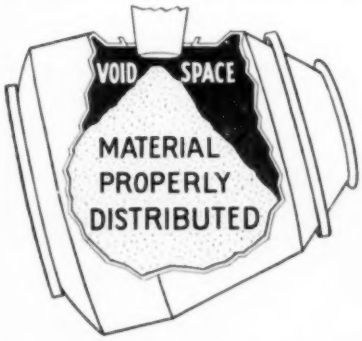
To perfect and prove the most efficient drum design for transit mixing, The Jaeger Machine Co. engaged Prof. S. C. Hollister (later President of the American Concrete Institute) to study all factors. The Hollister Tests, covering more than a year and including over 2000 cylinders plus wet analysis studies, not only determined the present shape and mixing blade arrangement of the Jaeger drum but also demonstrated the absolute necessity for Throw-Back Blades to obtain a satisfactory end-to-end mix of materials regardless of slump. The positive reversing action of these blades, as used exclusively by Jaeger, explains in large measure the proven higher quality of concrete and faster mixing action produced by the Jaeger drum.

Jaeger has applied the principles demonstrated in the Hollister Tests to every later model developed to meet new specifications—a modern truck mixer tested to a scientific standard.



EFFICIENT MECHANICAL MIXING results from this exclusive combination of 5 factors: Short, large diameter drum, 45° cones at both ends, continuous spiral mixing blades (A), throw-back reversing blades (B), and dual revolving water sprays (C) providing rapid, uniform distribution of water from end-to-end of drum. Comparisons with official studies prove that 40 revolutions of this drum produce a more homogenous concrete than is obtained in their normal mixing time by standard paving mixers.

(2) TOP LOADING—Advocated by Jaeger as the ONLY Practical Insurance Against the Abuse of Over-Loading (95% of All Jaegers are Exclusively Top Loading)



THE RIGHT WAY TO LOAD HIGH DISCHARGE DRUMS

Fully batched in one quick drop. Materials properly distributed. Ample void space remains—a factor of safety to insure proper mix irrespective of gradation, consistency of mix or the variations due to the human element.

At one time the Jaeger tilting drum building mixer, loading and discharging thru the same opening, represented a business of over $2,500,000 a year. It later fell into disfavor with engineers and was banned by more than 40 State Highway Depts. for only one reason—the ease with which such a drum can be loaded beyond good mixing capacity. For this reason we abolished our original end-loading type of truck mixer in 1930 in favor of a top loaded drum which, because of the coneing of materials, automatically insures the ratio of void space needed for proper mixing.

Since 1930 all standard Jaeger Truck Mixers or Agitators, whether conventional or high discharge type, have been exclusively top loading. In 1940 we substantially increased drum sizes to permit ribbon loading thru the top to the full capacity of present day trucks, thus further discouraging use of end charging devices for overloading.

Specifically for old plants or temporary bins not equipped for modern ribbon loading, Jaeger provides this end loading attachment. Its use is limited.

THE JAEGER MACHINE COMPANY, COLUMBUS, OHIO

Distributors in Over 120 Cities

TDA*

A CATALYST DISPERSING AGENT

We are taking this opportunity to present to the members of the American Concrete Institute some hitherto unpublicized facts pertaining to the beneficial effects of TDA on the quality of concrete. For the past eight years TDA has been recognized as a powerful grinding aid and has been used largely for this purpose by the cement industry. However, a few large cement consumers have as a result of their investigations, recognized its quality benefits and specified it. In 1941 alone, a total of over twenty million barrels of portland cement were ground with this catalyst-dispersing agent. We feel, therefore, that it is of timely interest to the concrete engineer to know what TDA is and why it is instrumental in improving the quality of concrete.

TDA is a catalyst and dispersing agent for portland cement. By definition, a catalyst increases the rate of chemical reaction without itself being consumed. TDA as a catalyst increases the reaction rate between portland cement and water without changing the end products of the reaction. The catalytic effect of TDA is enhanced by the dispersing action of TDA which, by minimizing agglomeration of the particles of cement, exposes more cement surface to reaction with water. Since dispersion is a purely physical phenomenon, the end product of the reaction is again unchanged.

How TDA works is illustrated by the microphotographs in Figure 1, which were taken of cement paste at the end of one hour and fifteen minutes

BLANK TDA

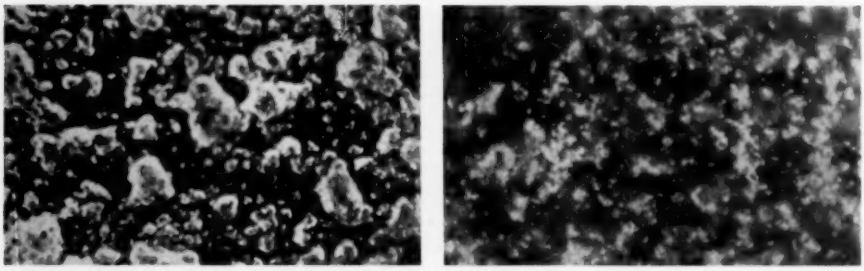


FIGURE 1—Microphotographs of cement pastes, with and without TDA, made with a Leitz Ultropak reflecting light microscope. Paste 24% N. C.

*TDA is a mixture of triethanolamine salts and highly purified soluble calcium salts of modified lignin sulfonic acid.

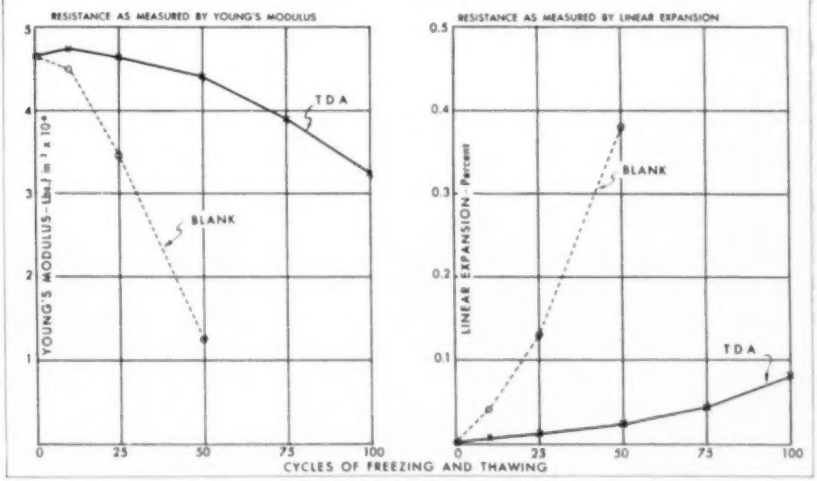
of hydration. Note that the blank cement paste shows a large number of dark spaces between the agglomerates, indicating voids due to lack of dispersion, with gelation just around the surface of the agglomerates. In the case of the TDA cement paste, however, water voids are negligible with practically complete gelation of the individually dispersed particles of cement as indicated by the absence of dark areas.

From the foregoing theoretical considerations, it follows logically that the combined effect of catalysis and dispersion produces more homogeneous, less permeable concrete having greater resistance to weathering. The fact that a better concrete does result from the use of TDA is indicated by the data given in Figures 2, 3 and 4. These are typical average results. Furthermore, this is accomplished at no sacrifice of other desirable qualities of concrete such as strength, bond to steel and density.

Due to lack of space, it is impossible to present in full the substantiating data obtained by many outstanding laboratories in the United States. We invite anyone who is interested in a particular phase or phases of the data to write us for further information.

FIGURE 2–EFFECT OF TDA ON DURABILITY OF CONCRETE

(Treated and untreated cements made from same clinker)



As already noted we have until recently advocated TDA largely as a grinding aid and have given little emphasis to its beneficial effect on quality. The optimum amount of TDA required as a grinding aid has been controlled entirely by the economics of the case, the main objective being to effect a decrease in grinding cost. The intergrinding of TDA for this purpose

increases mill production rate as shown in Figure 5 but also has the incidental advantage of making the TDA an integral part of the finished cement so that its use entails no added care at the concrete mixer.

Now however, some consideration is being given by the large cement users to the possibilities inherent in the use of cement additions, particularly the foam producing types which entrain excessive air, to implement

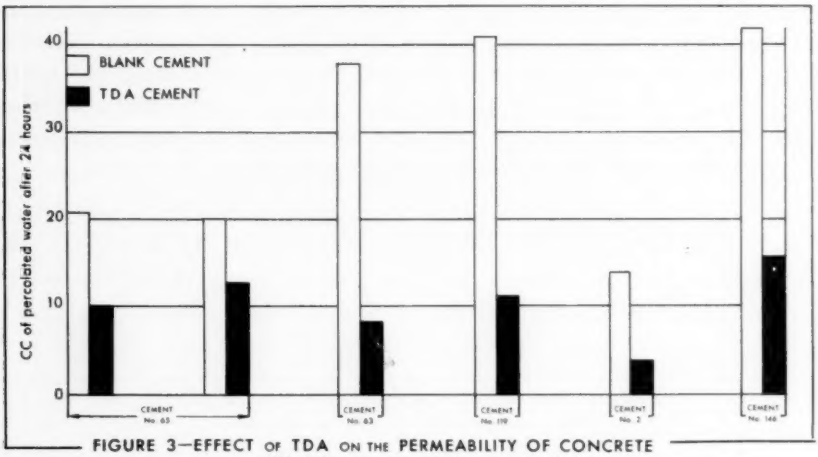


FIGURE 3—EFFECT OF TDA ON THE PERMEABILITY OF CONCRETE

NOTE: *Surface areas of comparative samples are identical*

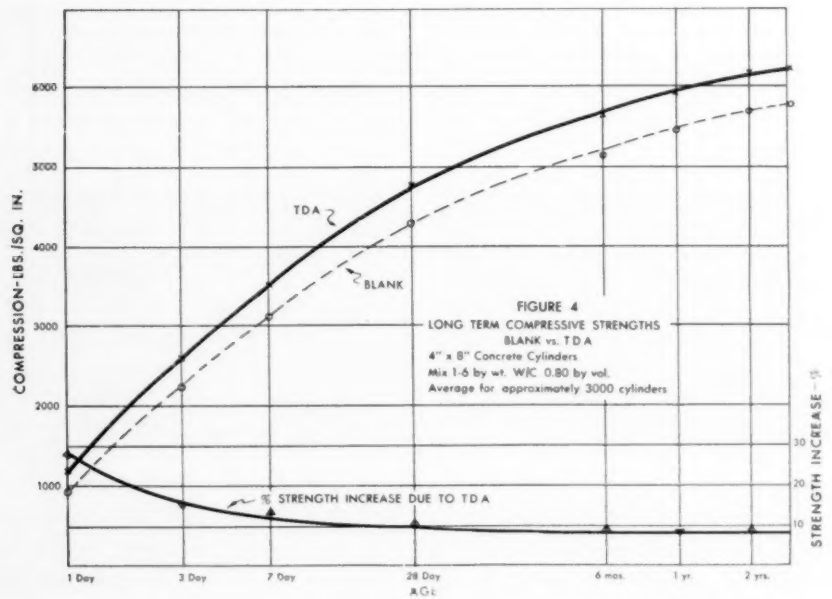


FIGURE 4
LONG TERM COMPRESSIVE STRENGTHS
BLANK vs. TDA
4" x 8" Concrete Cylinders
Mix 1-6 by wt. W/C 0.80 by vol.
Average for approximately 3000 cylinders

their usual techniques and procedures in order to obtain increased workability, as for example in pumped concrete. In this connection, TDA when interground in the amount of ¾ lb./bbl. of cement, will give unparalleled increases in workability with no sacrifice in strength or bond, or without

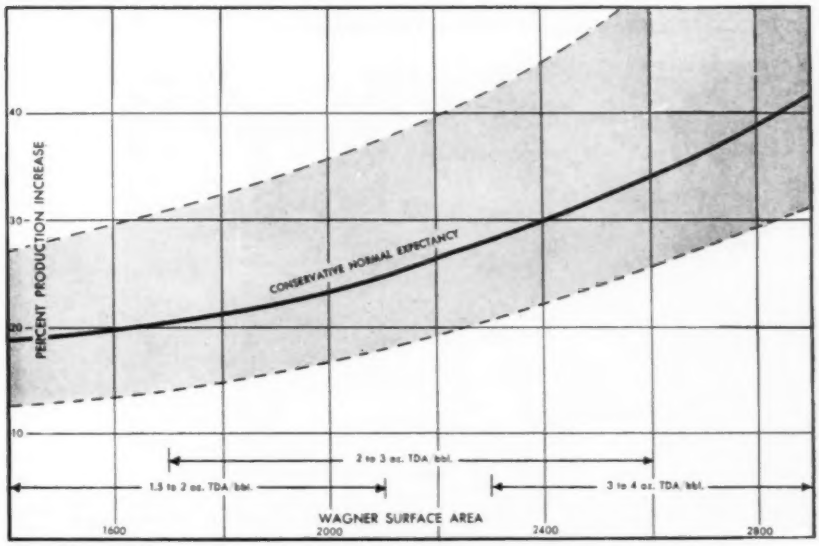


FIGURE 5—TDA PERFORMANCE CURVE

entraining the excessive air, the other qualities being still further improved. The use of this amount of TDA at a cost of 6-8 cents per cubic yard will in general reduce the water cement ratio 15%.

Since TDA is a product of research, we feel that the reader will be interested in the cement and concrete laboratory of the Dewey and Almy Chemical Company in which TDA was developed and which is open to members of the American Concrete Institute and other concrete engineers. It was in our laboratory that the possibilities of applying the principle of colloidal dispersion to portland cement were first investigated, which investigation finally culminated in the combining of catalysis and dispersion in order to obtain best results.

The physical testing laboratory and the concrete curing room are illustrated in Figures 6 and 7. The laboratory covers 4000 square feet of floor area, and is air conditioned throughout to assure uniformity of results. It contains the most modern equipment. The method for subjecting specimens to alternate cycles is unique in that various full size rooms are available having temperatures of +32, +5, —5 and —20°F.

At present more than 50% of the laboratory's time is devoted to pure research, including over two years of investigation of various air entraining agents. This research has in many instances resulted in the development of new apparatus to meet special conditions such as the latest innovation, a workability apparatus for tremie concrete.

FIGURE 6—Part of Physical Testing Laboratory

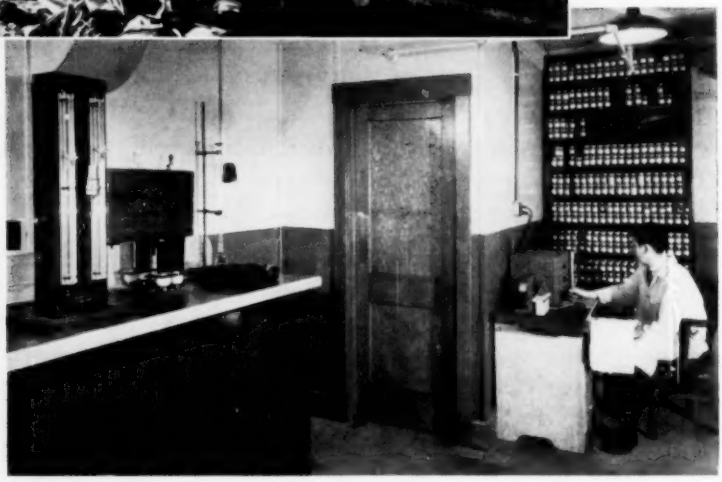


FIGURE 7—Part of Instrument Room Showing Sonic Vibration Apparatus

However, TDA has long since passed the experimental stage as indicated by the cumulative sales curve given in Figure 8. At this time TDA is the only material permitted without restriction in the manufacture of portland cement under the American Society for Testing Materials specifications. Its acceptance by the cement consumer is attested to by the recognition accorded

it by the largest organized users of portland cement in the United States. For example, a large percentage of the cements used in Grand Coulee, Marshall-Ford, Shasta, Mahoning and Loyal Hanna dams was interground or is being interground with TDA either by permission or specification. TDA's performance has justified the recognition given it.

We wish to emphasize that TDA is not a cure-all. It will not compensate for improper workmanship, poor design of mix, or slipshod control. It will however, produce varying degrees of improved workability, depending upon the amount of TDA used, without sacrifice of strength, bond to steel,

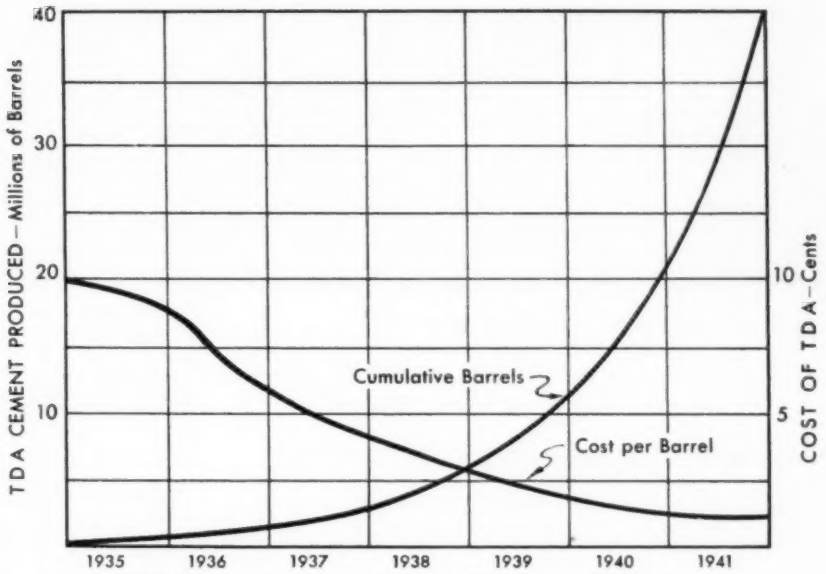


FIGURE 8 — Cumulative Barrels of TDA Cement Produced

or other desirable properties of concrete. It will, even in any of the lesser amounts commonly used as a grinding aid, increase the durability of structural concrete.

●

DEWEY AND ALMY CHEMICAL COMPANY

CAMBRIDGE, MASSACHUSETTS

THIN SHELL CONCRETE ROOFS

LESS STEEL with BALANCED DESIGN

Total weight of reinforcing only about 1/3 to 1/2 of amount required for comparative structural steel design.

CONSTRUCTION SPEED and LOW COST

Smooth underside of roof permits formwork to roll on own wheels for many reuses and remarkable savings in cost.

GREATER SPANS for MORE USEFUL SPACE

Beam action of curved roof utilized to save materials. Fewer columns and improved lighting and ventilation.

ADAPTABILITY and PERMANENCE

Defense plants, warehouses, hangars, etc., built with speed and economy. Less vulnerable to bombing, sabotage and fire.

ROBERTS AND SCHAEFER COMPANY

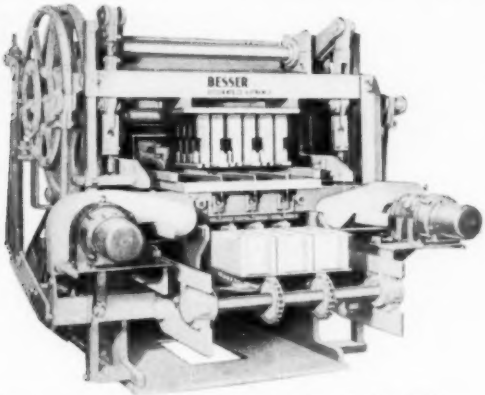
ENGINEERS

307 North Michigan Avenue, Chicago, Ill.

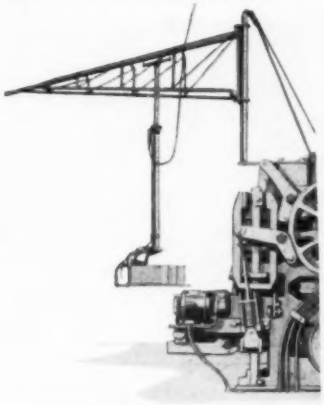
Eastern Branch Office: 2017 "S" Street, N.W., Washington, D. C.

BESSER VIBRAPAC PLAIN PALLET STRIPPERS

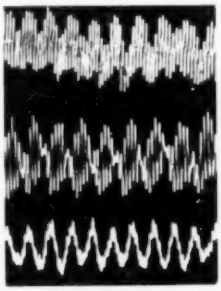
Undirectional Vibration for compacting concrete blocks (Flam and Besser patents) combined with Besser patented principle of making all sizes on one set of plain pallets.



Besser Super 100% Automatic Plain Pallet Vibrapac. Capacity 600 8" x 8" x 16" blocks per hour, made 3 at a time on one plain pallet. Smaller units made in multiples of 4 to 32 (brick) on the same pallets.



Power offbearing hoist — One man offbears 600 8" blocks per hour. No lifting—Simply guide the hoist.



Vertical

Side-wise

Back and Forth

Scientific Recording of undirectional vibration as utilized under Flam and other patents, and patents pending, on Besser Vibrapac Plain Pallet Strippers.

Photograph shows vibration impulses for one second as recorded by light beam on motion picture film, magnified nine times.

Undirectional vibration under Flam patents was used to make the first vibrated blocks ever made on a production basis and sold commercially.

In the following localities from one to seven Besser Vibrapac equipped plants may be seen in production operation. Names of nearest operators and literature on request.

ALABAMA
Birmingham
Alabama City

CALIFORNIA
Napa
Los Angeles

CONNECTICUT
South Norwalk

DELAWARE
Wilmington

DIST. OF COLUMBIA
Washington

FLORIDA
Miami

ILLINOIS
Chicago
Lockport
Champaign
Peoria
Rockford

INDIANA
Indianapolis
Hammond
Gary

IOWA
Davenport

KENTUCKY
Louisville

LOUISIANA
Baton Rouge
Bossier City

MARYLAND
Baltimore
Bladensburg

MASSACHUSETTS
Watertown
Medford

MICHIGAN
Detroit
Lansing
Alma

MINNESOTA
Minneapolis
Rochester
Duluth

MISSOURI
St. Louis
Kansas City

NEBRASKA
Omaha

NEW JERSEY
Morristown
Lyndhurst

NEW YORK
New York City
Syracuse
Albany
Rochester
Auburn
Buffalo

OHIO
Columbus
Cincinnati

OHIO—Contd.
Cleveland
Toledo
Lima
Springfield

OKLAHOMA
Lawton

PENNSYLVANIA
Philadelphia
Pittsburgh

TENNESSEE
Daisy
Jackson

VIRGINIA
Richmond

WASHINGTON
Seattle

WEST VIRGINIA
Charleston

WISCONSIN
Milwaukee
Racine
Menasha
EauClaire

CANADA
Vancouver
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A Five Year Service Record

REILLY PLASTUVIA FILLER

Does Not "Pull" in Summer nor Chip in Winter

● After five years of service in the busiest street in downtown Indianapolis—under alternating stress of severe winter and summer weather conditions—the PLASTUVIA joints in this concrete pavement are still tight and unbroken. There has been no pulling away from the concrete—no chipping—no shrinking—no cracks.

PLASTUVIA is a waterproof coal tar filler which bonds firmly to brick and concrete, resists traffic pull, and permanently fills and seals the openings between surfaces to prevent water damage. PLASTUVIA meets every requirement of a serviceable filler for concrete, brick or stone road joints and cracks.

Further information on PLASTUVIA — also on REILLY Concrete Curing Compound — will be sent on request.

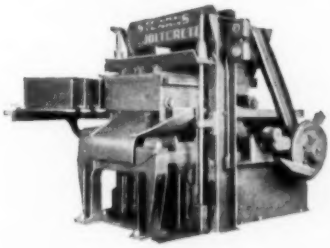
Reilly
COAL TAR PRODUCTS

REILLY TAR & CHEMICAL CORPORATION

Executive Offices: Merchants Bank Building, Indianapolis, Indiana

2513 S. DAMEN AVENUE, CHICAGO, ILLINOIS 500 FIFTH AVENUE, NEW YORK, N. Y. ST. LOUIS PARK, MINNEAPOLIS, MINN.

FIFTEEN · PLANTS · TO · SERVE · YOU



Improvements in the Stearns Joltcrete Block Machine that result in four distinct advantages

The Wisconsin University tests* reported in 1939 clearly demonstrated the superiority of Stearns vibrated concrete masonry wall units over similar products of tamped concrete. All of the vibrated units tested were made on the Stearns Joltcrete Machine.

New Base Design

To add to the recognized Stearns Joltcrete principle of limited amplitude vibration—vibration under pressure—we are able now to announce a refinement in machine design that preserves all the advantages gained by limiting the up-stroke of the mold box and develops important new ones.

Stearns engineers have introduced a resilient element in the base construction and have tested it out for months on machines operating in the field.

All new No. 9 Joltcrete machines have this new base construction and, in accordance with established Stearns policy, the improvement is made available for ready application to such Joltcretes already in service.

This new feature is reported by concrete products manufacturers to have these advantages:

Four Important Results

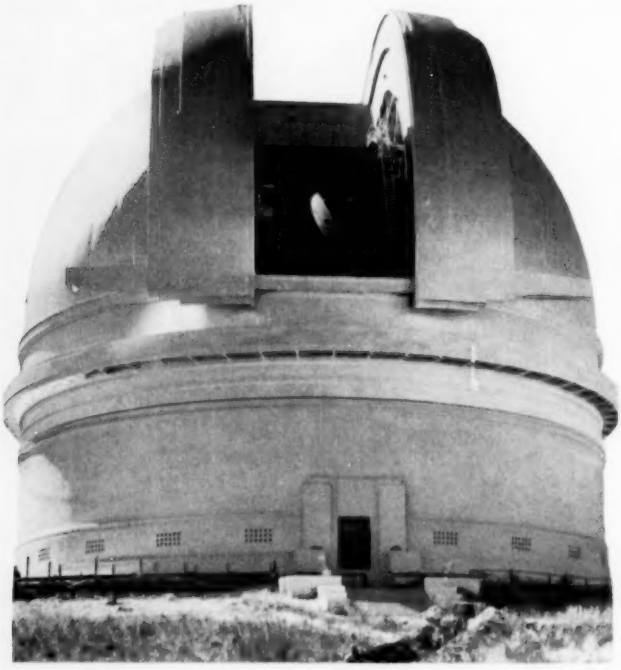
1. A denser block—aggregates are consolidated to a greater degree, making for greater block strength, greater moisture resistance.
2. Machine parts under vibration have longer life, hence replacement cost is reduced.
3. Need for adjustment of amplitude is less frequent—fewer interruptions of production.
4. Quieter operation.

Low Initial Investment

Stearns Joltcrete Machines not only make superior wall units, but can be equipped for the economical production of miscellaneous shapes, such as the rip-rap units made for the Kingsley Irrigation Dam at Ogallalla, Nebr. All units made on the Joltcrete are low in cost not only because of the speed and economy of operation but also because of lower overhead, as the investment in equipment is in many cases only about half that required for other equipment of comparable productive capacity.

*See "Tests on Concrete Masonry Units Using Tamping and Vibration Molding Methods", JOURNAL of American Concrete Institute, Nov. 1939.

Stearns Manufacturing Co., Inc., Adrian, Mich.



Solved
the VIBER WAY

"200-Inch" Astrophysical Observatory of the California Institute of Technology, Palomar Mountain, California

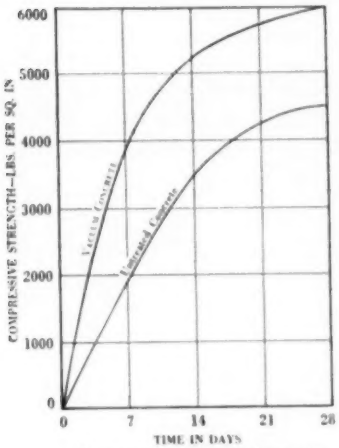
VIBRATION, the most destructive force known to nature, has now been made constructive by VIBER. The experience gained in concrete vibration* has led to the development of our present line of internal vibrators so successfully used all over the world.

**First used on the Santa Barbara Courthouse in 1926.*

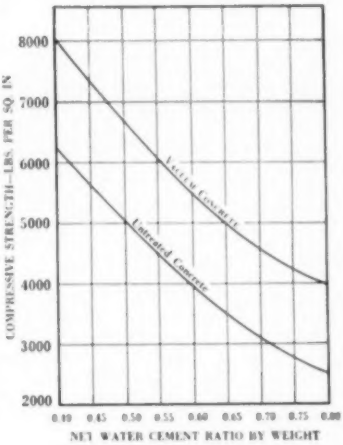
The Flexible Electric Vibrator, illustrated, is one of a full line of flexible and semi-rigid vibrators—electric, pneumatic or gasoline powered—built for every type of job.

VIBER COMPANY
CONCRETE VIBRATORS
726 SOUTH FLOWER STREET BURBANK, CALIFORNIA

VACUUM CONCRETE



Strength-Age Relationship



Strength-Water Relationship

Significant curves for plain and Vacuum-Processed Concrete developed by U. S. Government Tests

VACUUM CONCRETE Process is a simple, practical method for removing from concrete, after it is placed, from 5 to 8 gallons of water per cubic yard.

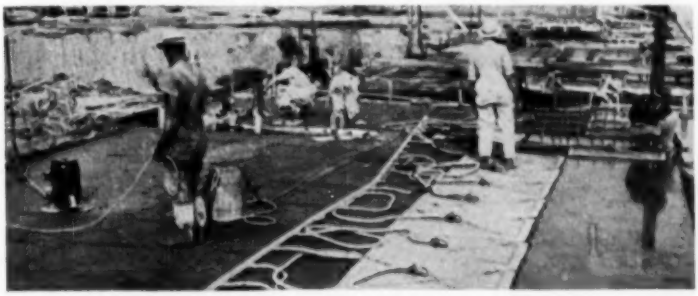
METHOD OF OPERATION. As soon as one end of a pour is concreted, a line of 3′ x 4′ suction mats is placed as shown in photograph below, and connected to a mobile Vacuum pump capable of handling up to 1000 cu. ft. of air per minute at 20″ vacuum. Evacuated water is trapped for disposal before reaching pump.

After 4 to 10 minutes (depending on thickness of slab) concrete is hard enough to walk on, and the entire line of mats is moved ahead, progressively covering and exhausting excess water from the entire floor area. Finishing can be started immediately behind mats.

EFFECTIVENESS. Process is effective for full depth of slabs of normal thickness—4″ to 12″.

ADVANTAGES in Quality, Speed and Economy can be demonstrated.

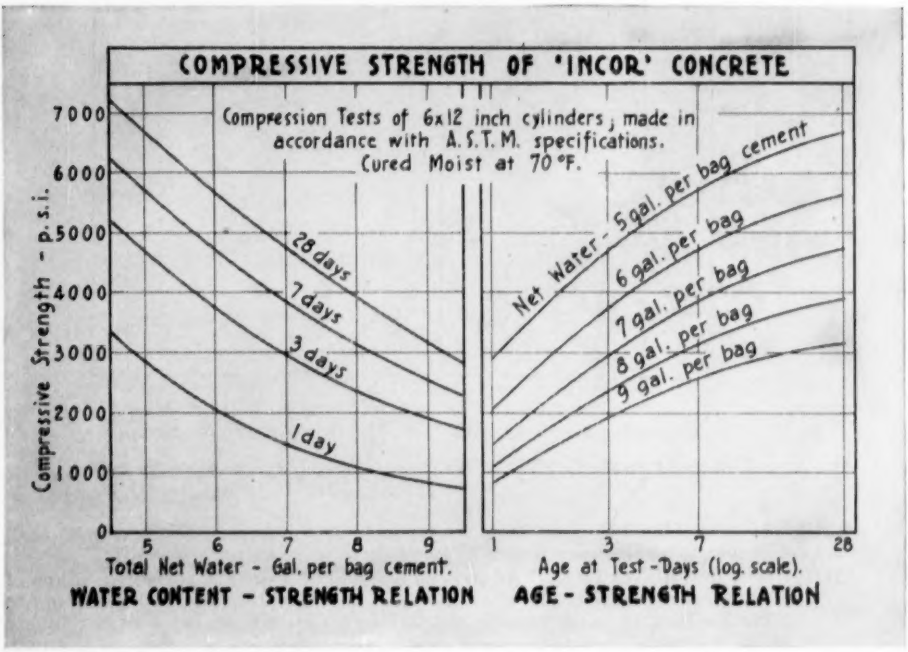
VACUUM CONCRETE Process has been used on more than 20,000,000 sq. ft., of slab in buildings constructed under the supervision of U. S. Navy, War Department, Federal Works Agency, U. S. Housing Authority, and District of Columbia.



VACUUM CONCRETE, INC.

4210 Sansom Street, Tel. BARing 4545 Philadelphia, Pa.

15-YEAR PERFORMANCE RECORD



BASIS OF PLANNED CONCRETING: Time required to attain service strength determines the number of form sets needed for desired erection speed. By estimating concrete costs in terms of time and forms as well as cement, it is easily possible to select cement to fit the job—using 'Incor' or Lone Star, whichever shows the lower cost under given job conditions. This is planned concreting—and it saves both time and money.

PROVES BASIC ADVANTAGES OF

Planned Concreting

SELECT CEMENT TO FIT THE JOB

FIFTEEN years ago it took concrete a week or more to attain service strength. In 1927, 'Incor' 24-Hour Cement was introduced, eliminating unproductive time waiting for concrete to harden.

Mixed with 6 gal. water, 'Incor' produces 2000-lb. concrete in 24 hours. That permits early stripping and faster form re-use, cutting form costs 50% or more. 'Incor' also simplifies winter construction, saving 60% in heat-protection expense and reducing freezing risk. *Concrete becomes a ready-to-use, year-around building material.*

'Incor' produces strong, watertight, wear-resistant concrete with only 24 to 48 hours curing. Fifteen years' field performance has demonstrated durability and high ultimate strength as well as reliable high early strength.

Compare 'Incor' savings against its extra cost. Select the cement which shows the lowest concreting cost under given job conditions. Let form, time and curing costs—and, in winter, heating costs—determine whether to use Lone Star or 'Incor'*.

Construction economies—both time and money—are all-important. Planned concreting saves days—and dollars. *Reg. U. S. Pat. Off.

SYMBOL OF QUALITY EVER SINCE 1900

LONE STAR CEMENT CORPORATION

342 Madison Avenue, New York

Offices: Albany • Birmingham • Boston • Chicago • Dallas
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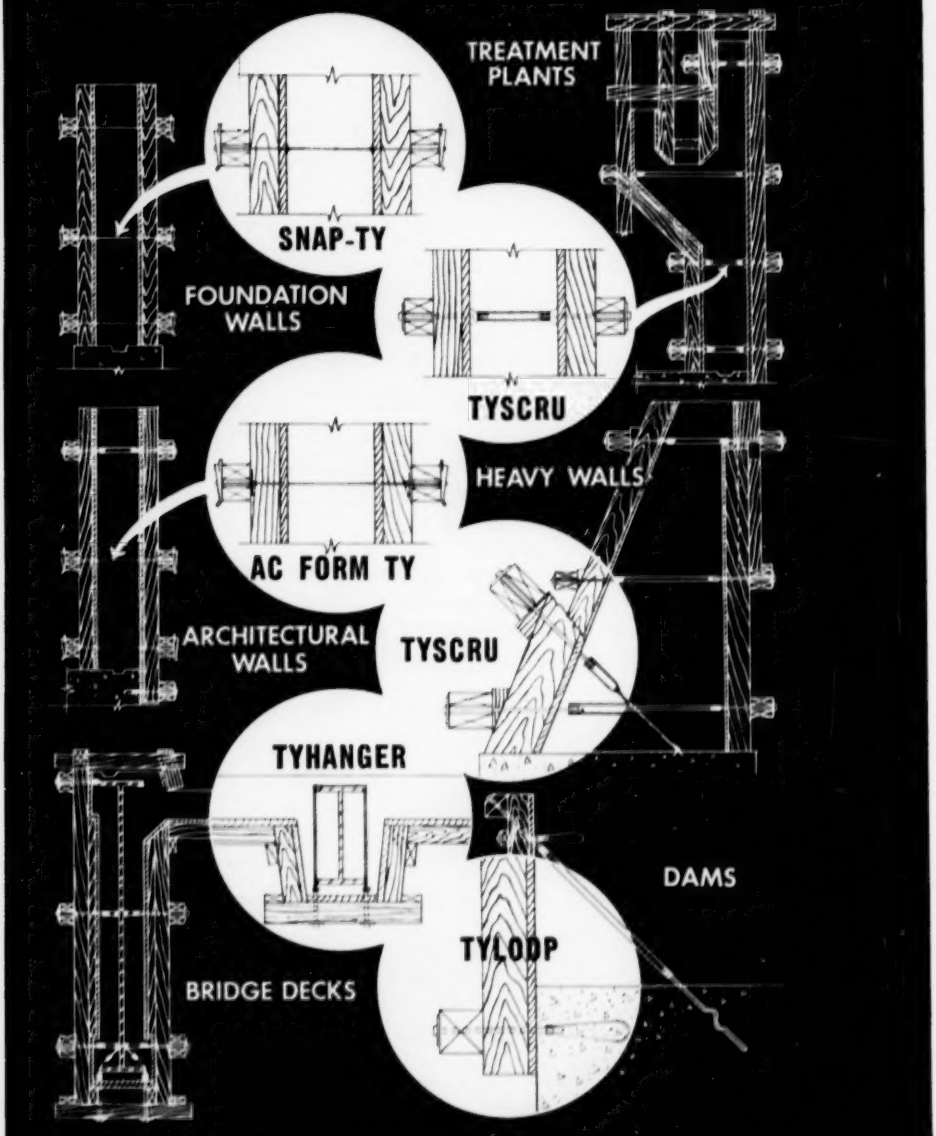
RICHMOND PRODUCTS

CORRECTLY ENGINEERED TO SAVE TIME AND LABOR ON YOUR JOB!



JOB ESTIMATES & CONSTRUCTION PLANS SUPPLIED · WORKING PARTS LOANED

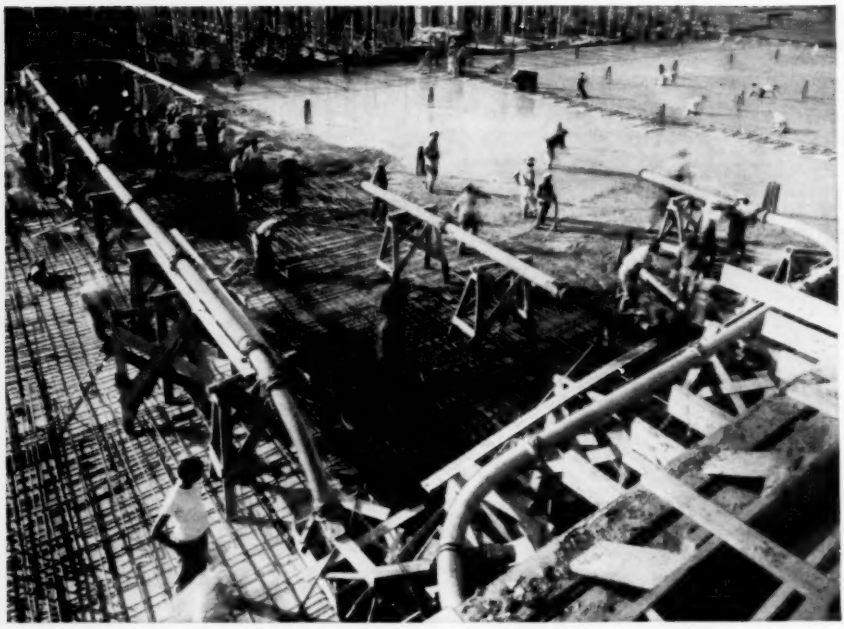
FOR CONCRETE FORM-WORK

	TIE	TYPE		SAFE LOAD	USE
TYING DEVICES	SNAP-TY		CONE	3000 #	Unfinished Foundation Walls
			1" Break	3000 #	Finished Foundation Walls
	AC FORM TY		PULLOUT	3000 #	Exposed Architectural Concrete Walls
			BREAK BACK (1½")	3000 #	
	TYLOOP		½ x 4"	3000 #	Miscellaneous Tie used with #9 Wire, 6 strands for ½" — 10 strands for ¾"
			¾ x 6"	5000 #	
	TYSCRU		½"	5000 #	Heavy Foundations—Small Bridges
			½" Waterseal	5000 #	Watertight Structures
			¾"	9000 #	Large Bridge Structures
			1" - 2 Strut	14000 #	Massive Piers & Dams
			1" - 4 Strut	25000 #	Steel or Heavy Timber Form Work
ANCHORING DEVICES	TYLOOP		½ x 4"	1500 # *	Form & Scaffold Anchorage
			¾ x 6"	3000 # *	Form & Scaffold Anchorage
			¾" Flared 2 Strut	9000 # *	Light Lift Form Anchorage
			1" Flared 4 Strut	14000 # *	Heavy Lift Dam Form Anchorage
	CRIMPED ANCHOR DOWEL		½ x 22"	5000 # *	Tyscru Anchorage to Concrete
			¾ x 27"	9000 # *	Tie-Downs to Concrete
			1 x 32"	14000 # *	Lift Form Diagonal Tie Anchorage
	SWEDGED 15° ANCHOR DOWEL		½ x 28"	5000 # *	Tyscru Anchorage to Rock
			¾ x 33"	9000 # *	Lift Form Anchorage to Rock
			1 x 38"	14000 # *	Tie-Downs to Rock
	RICHMOND SCREW ANCHOR		1"	14000 # *	Invert Form Anchorage
			1¼"	21000 # *	Heavy Form Anchorage
			1½"	32000 # *	Cantilever Form Anchorage
HANGING DEVICES	TYHANGER		½"	6000 # **	Heavy Slab Form Support
			¾"	10000 # **	Bridge Deck Form Support
			1"	20000 # **	Pier & Dock Form Support
	HANGER-TY		½"	1800 #	Fireproofing Hanger
			¾"	2500 #	Light Deck Form Hanger
	HANGER FRAME		½"	6000 # **	Flat Slab Form Hanger
	FASCIA HOOK		½"	800 # ***	Fascia Form Tie

* 2000 # Minimum concrete assumed. ** Total load on both legs of hanger. *** Optional at 1200 #.

RICHMOND SCREW ANCHOR CO., INC.

816-838 LIBERTY AVENUE · BROOKLYN, NEW YORK



Now, more than Ever
WAREHOUSES BY PIPELINE!

Concrete is pumped in place by two Rex Pumpcretes on the floors for a new United States Quartermaster's depot—a job entailing 52,000 cu. yds. of Pumpcrete-pumped concrete!

WE HAVE always said that the Rex Pumpcrete was a "natural" for most concrete jobs—because it simplifies the job of placing concrete in difficult form work—because Pumpcrete permits a snappier, more orderly job progress—because Pumpcrete saves the money and time formerly spent in erecting and operating buggy runs, chutes, hoists, towers and other old-fashioned equipment.

Today, speed is the essence of every job—and speedy placement of concrete is one of Pumpcrete's biggest advantages—placing on many jobs up to 65 cu. yds. per hour with one machine.

Before you bid on any defense contract, take a look at Pumpcrete. See it first in the book, "Why and Where to Pump Your Concrete." To get your copy, simply write 1713 W. Bruce Street, Milwaukee, Wisconsin.

REX **PUMPCRETE**
THE PUMP THAT PUMPS CONCRETE
Sizes for Pumping from 15 to 65 cu. yds. per hour

CHAIN BELT COMPANY OF MILWAUKEE

Finishing Machines and Joint Installing Machines

RIBBON DUMMY JOINT SHOULD AND WILL LAST AS LONG AS THE ROAD—MAINTENANCE IS SELDOM (OR NEVER) NEEDED

Ask for detailed information

FLEXIBLE ROAD JOINT MACHINE CO.
WARREN, OHIO

PAPER 77—HIGH EARLY STRENGTH CONCRETE

RESEARCH and FIELD DATA

THE CALCIUM CHLORIDE ASSOCIATION, 4145 PENOBSCOT BLDG., DETROIT

The Use of Calcium Chloride as an Admixture in Concrete for High Early Strength and Low Temperature Protection

Calcium chloride incorporated in concrete, in accordance with the Standards of the American Society for Testing Materials, C82—that is, either in the dry form placed in with the aggregates, or as a solution included with the mixing water—offers particular concreting advantages. The principal advantages—attainment of equivalent early strengths in half the time ordinarily required, as well as greater ultimate strength—have been established through extensive investigations conducted by the National Bureau of Standards, Public Roads Administration, Portland Cement Assn., Ohio State University Experiment Station, and many state highway departments; together with over 25 years of practical field use. Supplemental benefits include improved workability, control of moisture, and volume changes.

In summarizing studies conducted on some 75 portland cements (standard, high early strength, white, and experimental—covering every practical range in composition) the National Bureau of Standards concludes: "Integral use of calcium chloride is effective in accelerating the curing of all cements . . . concretes with calcium chloride have greater strength than the plain concrete at all ages tested . . . calcium chloride not only greatly increases the early strength, but also appreciably increases the three year strength of the concrete."

Early Strength Concrete: At *one day* (70°F. curing) the average strength of concrete (1-2-4 by wt. 6½ gal. water per bag) with two per cent calcium chloride was 114% higher than the average strength of plain concrete, at *three* days this increase was 62%, and at *seven* days 29%. (Research Paper 782, National Bureau of Standards.)

Low Temperature studies on similar concretes stored at 20, 25, 32, and 40°F.temperatures immediately after molding were recently reported by the National Bureau of Standards to the Highway Research Board. Briefly these data in Table No. 1 show the following:

Plain concrete stored at *20°F.* although initially at a temperature of 70° was frozen and developed no strength throughout the 28-day test period; however, calcium chloride in concrete provided cold weather protection as evidenced by the consistent gain in strength.

At 25°F. the 2 and 3-day strengths of mixes containing calcium chloride approximated the 28-day strength of plain concrete.

At 32°F. the 1, 3, and 7 day strengths of mixes containing calcium chloride approximated respectively the 5, 7, and 28-day strengths of plain concrete.

At 40°F. the 1 and 3-day strengths of concrete containing calcium chloride respectively approximated the 4 and 7 day strengths of plain concrete.

The value of using calcium chloride when placing concrete during low temperatures may be appreciated when the rate of acceleration of strength development of the concrete is noted as shown in Table 1. This development of early strength in concrete with assurance of high ultimate strength is a most

important factor in concrete construction and of particular importance in the present defense program.

In addition to the early strength benefits from the use of calcium chloride it is also recognized that the ultimate strength of concrete as well is noticeably increased, as shown from the following table from the report of the National Bureau of Standards. (See Table 2.)

After years of investigation and tests by the California Department of Highways a report (Thomas E. Stanton, Nov. 1940, Journal Am. Con. Inst.) states: "With 2% Calcium Chloride the average strength increased from 4030 at 28 days to 6045 psi. at five years and 7700 pounds per square inch at 11 years" (126% of the standard). The report states: "Calcium chloride incorporated in the mix with the mixing water is apparently effective in maintaining a higher moisture content requisite for continued hydration and higher strength."

Use: At freezing temperatures and lower, the amount of calcium chloride used may be increased from two per cent by wt. of cement which is generally specified, to three or four per cent according to the intensity and expected duration of the low temperature. At temperatures below 70°F. 2 lb.; 70° to 90°F. 1.5 lb.; above 90°F, 1 lb.

Data. Additional data by Bureau of Standards, Highway Research Board, Portland Cement Assn. and others may be had in Bulletin No. 28 issued by The Calcium Chloride Assn., 4145 Penobscot Bldg., Detroit, Mich.

TABLE I—COMPRESSIVE STRENGTH OF CONCRETE

From the report of the National Bureau of Standards

Specimens: 6 x 12-in. cylinders; Mix: 1:2:4 weight: Water content 6½ gals. per sack.
Aggregate: Fine, Potomac River Sand; Coarse, Potomac River Gravel; Cement, Cement "C," Standard Portland

Temperature ° F.	Calcium Chloride Pct. Commercial	Compressive Strength in lbs./sq.inch				
		1 Day	2 Days	3 Days	7 Days	28 Days
20	0	0	0	0	0	0
	2	290	400	430	550	730
	3	320	480	710	820	1320
	4	500	700	780	830	1370
25	0	0	80	100	510	1130
	2	350	870	1250	1970	2470
	3	470	1030	1270	1990	2810
	4	550	1070	1310	2000	3030
32	0	100	210	470	1530	3150
	2	430	1030	1550	2660	3610
	3	750	1370	1690	2790	3730
	4	790	1380	1710	2890	3730
40	0	170	410	700	2100	3780
	2	950	1520	2130	2710	3980
	3	1180	1810	2220	2790	4110

TABLE II

INCREASE IN COMPRESSIVE STRENGTH OF CONCRETE DUE TO THE ADDITION OF 2 PERCENT OF COMMERCIAL CALCIUM CHLORIDE—From a Report by Paul Rapp

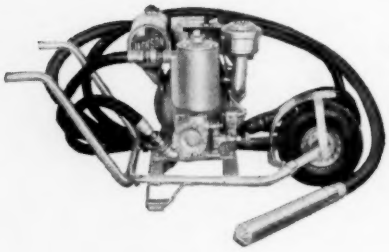
Age of Test	Portland Cement Assn.	District of Columbia	National Bureau of Standards
percent	*percent*	*percent*	*percent*
1 day		80	114
2 days	48	52	
3 days		46	62
7 days	25	21	29
28 days	7		14
90 days	7		7
1 year	12		8
3 years	9		

JACKSON VIBRATORS

DEPENDABLE EFFICIENT ECONOMICAL

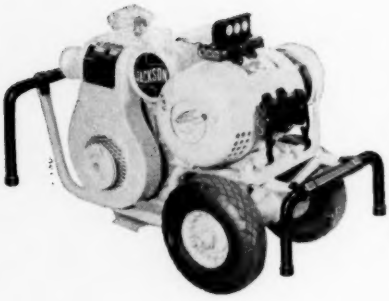
HERE'S WHY: Jackson Concrete Vibrators are the product of 24 years of experience in the manufacture of high frequency vibratory equipment. Improvements in design, materials, and construction have been made continuously in this

HS-A1 Hydraulic Concrete Vibrator



Operated by light oil pumped through neoprene lined, rubber-fabric covered hose line 34 feet long to displacement type hydraulic motor in vibrator head. Manual valve on power plant adjusts frequency, as desired, from idling to top speed of 7,000 V.P.M. No lubrication problem — all moving parts operate in the hydraulic medium (oil). Vibrator head 2¾" diameter, standard. Gas engine, air-cooled, 4.7 H.P. Choice of defense contractors — one hundred in use on Pacific Naval Air Base construction alone. Write for Form HYD-1141.

WS-4 Portable Power Plant



110 volt, 60 cycle A.C.—three phase or single phase. Capacity—2 K.V.A. (approx. 2000 watts). Husky, lightweight, small. Right for job lighting, running vibrators, saws, drills and like tools. No fussy controls or gadgets. Air cooled 4.7 H.P. gas engine — lots of power in a small package. One of several Jackson Portable Power Plants. Write for circular WS4-141.

SC-4A Vibratory Hand Screed

For slab, deck, floor, ramp, walk, or highway paving where use of a hand screed is indicated. Any wood plank can be used up to 20 ft. long. Handles quickly attached and detached, of welded tubular steel, cushioned to prevent transmission of vibration to operator. Vibratory motor mounted to side of plank as illustrated. Power required 110 or 220 volts, 3 phase, 60 cycle A.C. Specify voltage. Other types of screeds available for a variety of applications. Write for data.

ELECTRIC TAMPER & EQUIPMENT CO.

JACKSON VIBRATORS

DESIGNED RIGHT BUILT RIGHT WORK RIGHT

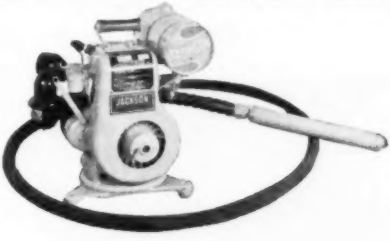
pioneer line. Thus, you can select or specify a type correct for your application with assurance of obtaining a vibrator that will perform properly under your job conditions. Your Jackson selection has qualified on many similar projects.

Vibratory Paving Tube—Models PT-200A and PT-200B

FOR SPEEDY FULL WIDTH AIRPORT AND HIGHWAY CONCRETE PAVING. Up to 25 ft. widths. Submergible dual tubes energized by powerful vibratory motors quickly transform harsh mixes to plastic state. Assures complete compaction—easy finishing. Cement savings up to 10% through reduction in W/C ratio. Attaches to any modern finisher. Variable frequency 3000–5000 V.P.M. Finger tip controls. Write for Jackson Vibrators, Inc. folder PT-440.

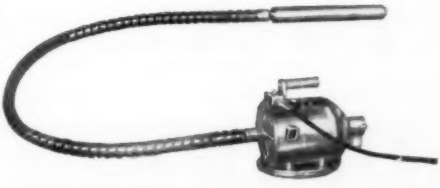
FS-6A Gas-Driven, Flexible-Shaft Concrete Vibrator

Fast. Driven by 3 H.P. air-cooled gas engine. Automatic clutch. V-belt drive. Countershaft has oilite bearings. Turntable base, dirtproof. Optional wheelbarrow with drop down lifting handles. Spring steel wire flexible shaft with hardened end fittings and drive. Rubber-fabric casing with synthetic rubber lining and flexible metallic liner. Streamlined coupling 3½" long—1¾" diam. of bar steel — cannot be broken or distorted. Vibrator head 2⅜" diam. x 18½" long or 1$\frac{13}{16}$" x 17". Frequency 7000–7200 V.P.M. variable with length of shaft and concrete consistency. Extra attachments. Write for Form FS-6A-1141.



FS-5A Electric-Driven, Flexible-Shaft Concrete Vibrator

Frequency 7000–7200 V.P.M. Shaft, coupling and vibrator head same as FS-6A. Powered by 110 volt A.C.–D.C. Universal motor, rated ¾ H.P. at 7000 R.P.M., fan cooled. Bowl type base, slides easily. Regular shaft lengths — 7, 14 or 21 ft. Request Form FS-5A-1141 for details.

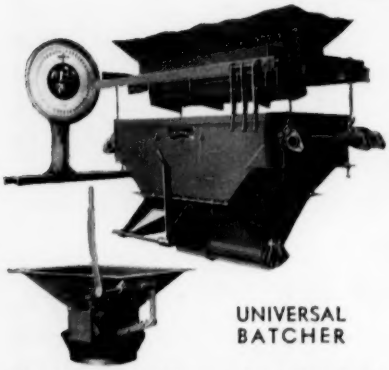


LUDINGTON . . . MICHIGAN

HELTZEL Accurate - Dependable Weighing Batchers Are Used for All Types of Concrete Construction . . . *Highways* . . . *Airport Runways* . . . *Concrete Buildings* . . . *Block Plants* . . . *Dams* . . . *Sewers* . . . *Tunnels* . . . *Subways* . . . *Arsenals, etc.* CAN BE ARRANGED TO DISCHARGE INTO BATCH TRUCKS . . . TRUCK MIXERS . . . AGITATORS . . . CONCRETE BUCKETS . . . STATIONARY MIXERS . . . PUMPCRETE HOPPERS, etc.

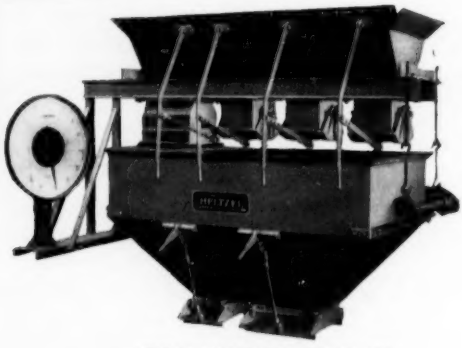
MANUAL - AUTOMATIC or INTERLOCKED CONTROLS

WRITE FOR COMPLETE INFORMATION AND DESCRIPTIVE LITERATURE



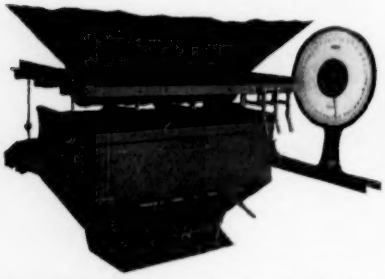
UNIVERSAL BATCHER

Interchangeable Hoppers for either Batch Truck or Mixer Charging.

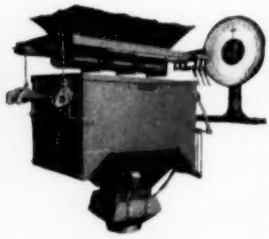


COMBINATION BATCHER

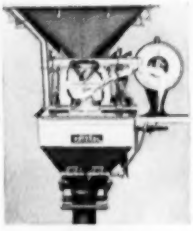
For Batching ALL Aggregate and Cement in One Operation.



2 or 3 Compartment Batcher Used where Specifications Call for Material to be Separated.



TRUCK MIXER BATCHERS
2 to 6 cu. yd. CAPACITY



HELTZEL DUSTLESS BULK CEMENT BATCHER

BULLETIN C-37

HELTZEL STEEL FORM & IRON CO.
WARREN, OHIO · U. S. A.

DURABLE CONCRETE FLOOR SURFACES

Recommendations and specifications for this type of floor call for the use of low water-cement ratio mixes using graded aggregates, which give high strength, water-tightness, resistance to wear, allow leaner mixes, and prevent segregation, assuring coarse particles at the surface to take the wear.

References are made to Institute Journals for March 1930, November-December 1934 and September 1938.

The Kelley Compactor Power Float is a piece of equipment designed to overcome the workability objection to the recommended, dry, harsh mixes. It delivers a dense, durable, dustless, hard wearing floor at a saving in labor and cement costs. (A Floor Specialist's letter follows).

THE VANNIER CO. INDUSTRIAL FLOORS BUFFALO, N. Y.

The old mix of one cement to two of coarse sand required from twelve to sixteen bags of cement per cubic yard, depending on the voids and bulking of the sand.

Mixes properly designed for power float method require approximately eight bags of cement per cubic yard.

The labor cost is at least one-third less, and this includes sufficient finishing labor to produce a smooth finish. For instance, in the past eighteen months I have done three jobs for nationally known concerns, for which I submitted two proposals, one on the specifications furnished, and one in accordance with our own specifications, with the understanding that we could not be responsible for the specified job, but we would be responsible for floors of our own design. In all three cases the price for our floors was 10% to 20% lower.

THE VANNIER COMPANY
C. B. Vannier

Write for "Concrete Facts about Concrete Floor Finishes" booklet and bulletin on Kelley Compactor Power Float to:

KELLEY ELECTRIC MACHINE COMPANY

287 HINMAN AVENUE BUFFALO, N. Y.

Cure Concrete With Hunt Process

The Hunt Process Company is proud of its record for 1941 when its products were used to cure concrete represented by more than 150,000,000 square feet of surface. This is positive proof that Hunt Process is a most satisfactory method for the curing of concrete. Further proof is the increasingly larger areas cured each year and the caliber of the engineers and architects who specify it. The results in actual construction have more than borne out the conclusions drawn from early laboratory tests and field study.

Experience has definitely shown that Hunt Process has the following features of particular interest to engineers and architects:

1. **Hunt Process reduces inspection and supervision to a minimum.**
2. **Hunt Process increases efficiency and speed of construction due to ease and convenience of application.**
3. **Hunt Process tends to prevent the hair checks and cracks which are due to stresses in the concrete surface resulting from the rapid and unequal evaporation of the mixing water while the concrete is still plastic and weak.**
4. **Hunt Process permits curing to proceed far beyond the usual "water curing" period.**
5. **Hunt Process insures full strength of the concrete throughout the mass to the very surface by retaining uniform water content.**

Hunt Process Company is keeping pace with the changes in concrete technique. A laboratory is maintained and constant investigation is being made of new materials and procedure so that the Hunt Process curing solutions will always be the best available. Our products are carefully checked to insure uniformity and quality. Hunt Process curing solutions are manufactured for the one and only purpose of curing concrete. They are the results of many years of laboratory investigation and experience on actual construction.

Membrane Curing Solutions Manufactured by Hunt Process Company

Hunt Process Black—A special blend of selected asphalts with pigment in a petroleum solvent.

Hunt Process Clear—A blend of oils and waxes in a petroleum solvent.

Hunt Process Clear White Pigmented—A special "clear" cure with white pigment for use on thin sections (i.e., canal linings) in the hot desert climates where temperature control is desirable.

Hunt Process is specified or used by many of the construction agencies represented by the following:

U. S. Army & U. S. Navy	State Highway Departments
U. S. Bureau of Reclamation	Counties & Cities
U. S. Public Roads Administration	Irrigation Districts
U. S. Indian Service	Industrial engineers and architects

Hunt Process Company operates generally only west of the Mississippi River.

Distributors in Dallas, Denver, Honolulu, Kansas City, Mo., Oklahoma City, Phoenix, Portland, Salt Lake City, San Francisco, Seattle, Vancouver, B.C.

Main Office and Plant

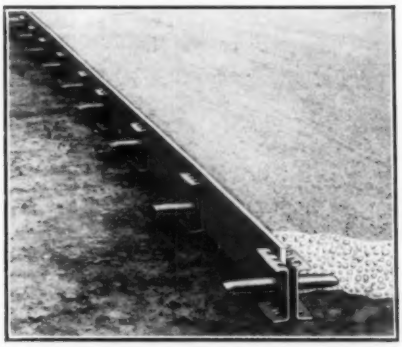
7012 Stanford Avenue - - - - - - - Los Angeles, California

HEL-CO *Modern Armor-Dowels*

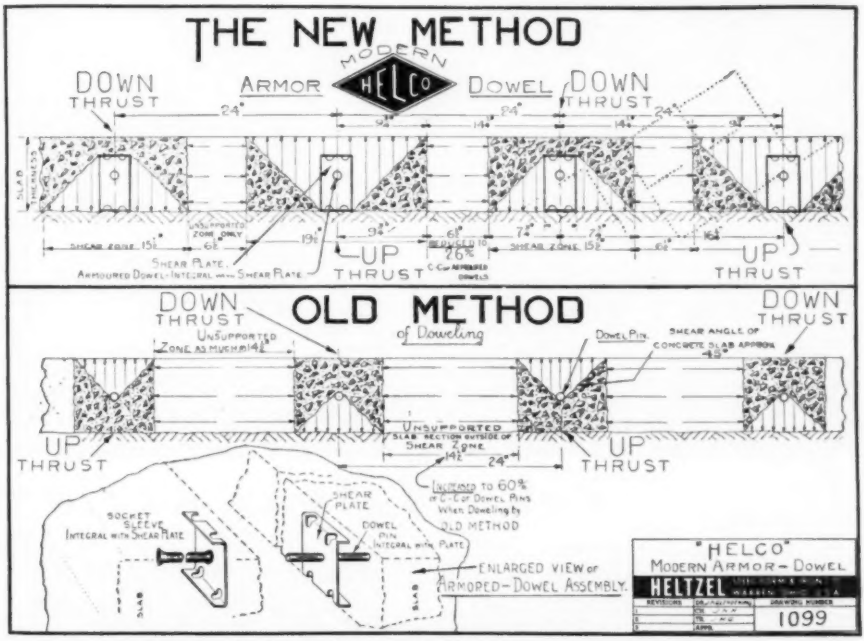
The NEW method of doweling for BOTH Longitudinal and Transverse Expansion & Contraction Joints in CONCRETE SLAB CONSTRUCTION . . . is already in use in State Highways and in Army airport paving and building floors.

SHEAR PLATE DESIGN GIVES 34% MORE SUPPORT THAN OLD METHOD
(See Drawing)

RIGHT: HEL-CO ARMOR-DOWELS in TYPICAL TRANSVERSE JOINT . . .

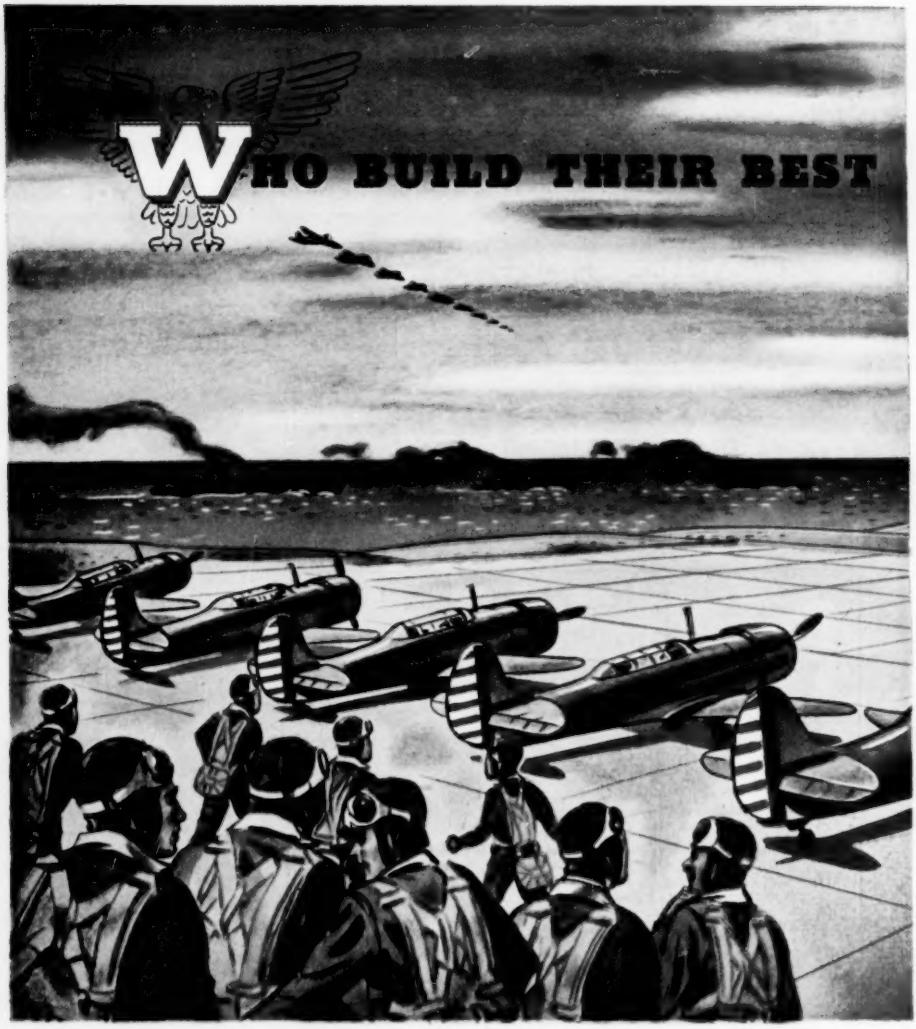


Write for Additional Information



LICENSED UNDER HELTZEL-WILLARD PATENTS . . .

HELTZEL STEEL FORM & IRON CO.
WARREN, OHIO · U. S. A.



CONTRACTORS PACIFIC

CONTRACTS NOy-3550

HAWAIIAN DREDGING CO., LTD.
P. O. Box 3468, Honolulu, T. H.
Mr. Walter F. Dillingham, Pres.

TURNER CONSTRUCTION CO.
420 Lexington Avenue, New York, N. Y.
Mr. J. Archer Turner, Pres.

RAYMOND CONCRETE PILE CO.
140 Cedar Street, New York, N. Y.
Mr. Maxwell M. Upson, Pres.

MORRISON-KNUDSEN CO., INC.
P. O. Box 1518, Boise, Idaho
Mr. H. W. Morrison, Pres.

FOR OUR DEFENSE . . .

have served their country well

It came at last—and in the back. But 'ere the bugle call had echoed through the land a million heroes arose to defend, to champion honor, decency and freedom—with a mailed fist. And there are other hands that serve, night and day. Strong, sure hands that build and fortify and mend . . . What time we have to build, no man can know. But build we will—and fast—lest brave men die for want of means to guard the land they love. And those who build their best for our defense have served their country well.

NAVAL AIR BASES

AND NOy-4173

J. H. POMEROY & CO., INC.
333 Montgomery St., San Francisco, Cal.
Mr. J. H. Pomeroy, Pres.

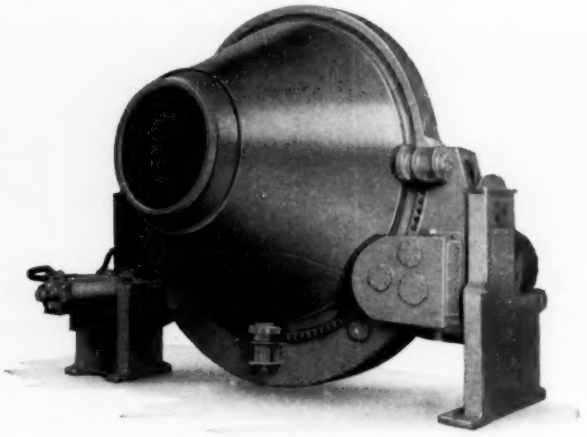
UTAH CONSTRUCTION COMPANY
1st Security Bank Bldg., Ogden, Utah
Mr. M. S. Eccles, Pres.

W. A. BECHTEL COMPANY
155 Sansome Street, San Francisco, Cal.
Mr. S. D. Bechtel, Pres.

BYRNE ORGANIZATION
P.O. Box 629, Old Army Base, Norfolk, Va.
Mr. John E. Byrne, Pres.

TILTERS

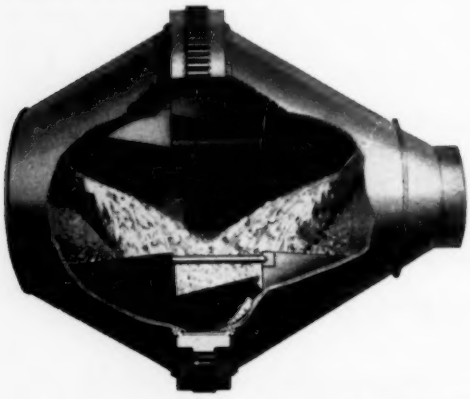
For 42 years, the Smith Tilting Mixer has set the pace for concrete mixers, earning a well-deserved reputation for fast loading, thorough mixing and rapid "Tilt and Pour" discharge. Today—the new Smith Tilter continues with that same basic design but with many additional refinements that result in even greater speed and operating efficiency.



View of a 2 yard Smith Tilter equipped with pneumatic tilting device. Also available in 1 yard, 3 yard and 4 yard sizes.

Famous "End-to-Center" Mixing Action

Smith Tilters produce specification concrete because of the duo-cone shape of the drum, plus the converging action of the V-shaped blades. These blades have only one function to perform, namely that of mixing. They catch the materials the moment they enter the drum and carry them toward the center of the drum. They cut through the materials—squeezing, kneading, compacting—and constantly fold the lowest layer of material over on to the top. The entire batch is continually pulled from the ends of the drum toward the center. Thus, the batch is always balanced in the center of the drum. And by keeping the materials away from the drum openings, spilling is avoided.



Large Aggregate

The Smith Tilting Mixer handles large aggregate without undue damage to the machine. Two conditions make this important feature possible. The drum openings are free from any discharge mechanism and the materials are NOT lifted to the top of the drum. Thus, the damaging action of the falling concrete is eliminated. Naturally, if large stones were used in the mix, such damaging action would be all the greater. Therefore, a tilting mixer, such as the Smith, is the ONLY concrete mixer which can, and regularly does, handle large aggregate successfully.

The T. L. SMITH COMPANY
2897 N. 32nd St. • Milwaukee, Wis.

SMITH MIXERS
USED ON THE WORLD'S GREATEST CONCRETE PROJECTS

VISIBLE MIXING

Permits Inspection of the Entire Batch Before Any of the Concrete is Discharged

Smith-Mobile's exclusive visible mixing feature permits consistency control through observation of the *entire* batch during the mixing process. It enables the contractor, inspector or foreman to determine when the batch is thoroughly mixed and of the proper consistency, *without having to discharge a part of the batch.*

The discharge door of any horizontal top loading mixer cannot be opened without discharging at least a part of the batch. And it's impossible to look into the drum of any competitive high discharge mixer to determine the characteristics of the concrete. However, on a Smith-Mobile Truck Mixer, the door can be opened wide, at any stage of the mixing cycle (even with a full load), and no concrete will come out until the drum is reversed. That's what makes real visible mixing possible.

The concrete manual of the U. S. Bureau of Reclamation (Second Edition, March, 1939) states that most transit mixers "cannot be operated in accordance with Bureau specifications, as no means are provided for directly observing the concrete as it is being mixed . . ." Only the Smith-Mobile has this very necessary advantage of visible mixing.

Efficient "End-to-End" Mixing Action

The distinctive shape of the Smith-Mobile drum, plus the scientifically designed spiral blades, produce a fast and thorough "End-to-End" mixing action. With the drum running in mixing direction, the blades force the aggregates which are below the top of the blade toward the closed end of the drum. The curved portion of the blade at the closed end of the drum forces the concrete in front of the blade to fold over and flow diagonally across the drum to the charging end. It's a real "End-to-End" mixing action, fast and thorough.

Smith-Mobile Truck Mixers make it possible for engineers to obtain truck-mixed concrete that meets the most exacting specifications as to aggregate distribution, sand-cement ratios, water-cement ratios and consistency throughout the batch.

Smith-Mobiles are available in five sizes: No. 2, No. 3, No. 3½, No. 4 and No. 5 truck mixers (2½ yd., 4 yd., 4½ yd., 5¼ yd. and 6 yd. agitators, respectively.)

The T. L. SMITH COMPANY
2897 N. 32nd St. • Milwaukee, Wis.

SMITH-MOBILE
THE ORIGINAL HIGH DISCHARGE TRUCK MIXER

HIGH EFFICIENCY IN READY MIXED CONCRETE PLANT

New speed and efficiency in ready mixed concrete operations were achieved by the U. S. Transit-Mix Concrete Corp. in supplying more than 200,000 cu. yd. of concrete for the Kankakee and Elwood Ordnance Plants. The batching plant was laid out as in the accompanying sketch, and was designed and manufactured entirely by the Butler Bin Company, Waukesha, Wisconsin, except for the belt conveyor.

Aggregates were shipped in on two railroad tracks, dumped into hoppers beneath the tracks, and carried by a horizontal feeder belt to a 36-in. inclined belt conveyor. A revolving turnhead at the top of the aggregate bin distributed the aggregates to the three compartments of the bin.

Cement, shipped in hopper-bottom cars, was fed by an undertrack screw conveyor into an enclosed bucket elevator, capacity 300 bbl. per hour, which discharged into an auxiliary hopper attached to the side of the aggregate bin. This hopper had two compartments, providing for standard and high early strength cement. Overflow spouts led to a low cement storage bin over the screw conveyor, from which the cement could be recirculated.

Below the aggregate bin was a 5-yd. four-beam weighing batcher for both aggregates and cement. While these were weighed manually, water was being weighed automatically in a 200 gallon water batcher. All materials were discharged through a collecting hopper, so that only one truck stop was necessary. A fleet of thirty 5-yd. truckmixers was needed for an average haul of 2½ miles. During winter operations aggregates were heated by

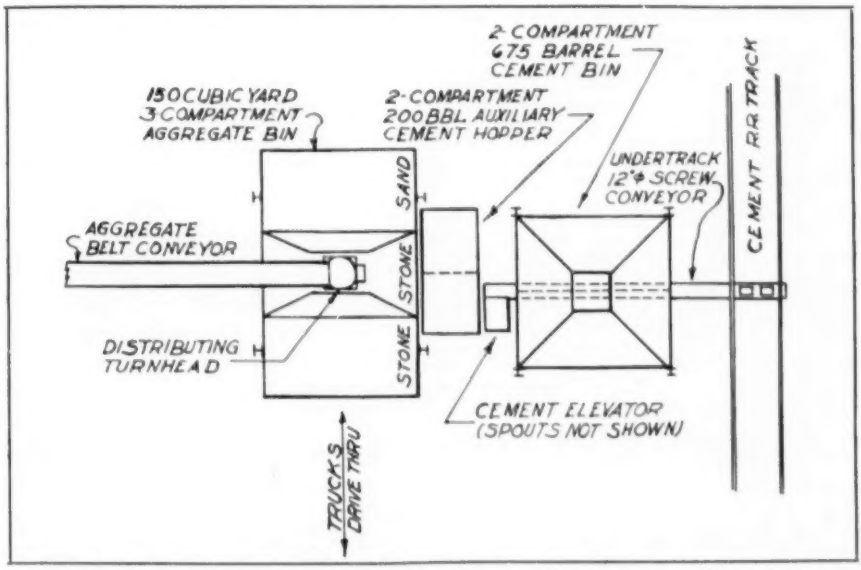
injecting live steam into the bin, for concrete was poured continuously at temperatures as low as 12 degrees below zero F.

Ten men, exclusive of truck drivers, formed the entire operating personnel of the plant: one batcher operator, one to handle cement cars, one belt conveyor operator, one to spot trucks beneath the plant, one to fire the boilers during cold weather operations, and five men to handle aggregate cars. Union regulations required the belt conveyor operator and the boiler man to be engineers; all others were laborers.

Complete figures on the production of this plant are not available, but a random selection of general contractor's receipts shows these pours:

April 28—1613 cu. yd. in 8½ hours (190 cu. yd. per hour)
April 29—1929 cu. yd. in 9½ hours (203 cu. yd. per hour)
May 2—1869 cu. yd. in 8½ hours (220 cu. yd. per hour)
May 15—2006 cu. yd. in 9 hours (223 cu. yd. per hour)
May 29—1782 cu. yd. in 8½ hours (210 cu. yd. per hour)

These do not represent the maximum capacity of the plant, but merely what it was called upon to produce on these days. It is stated that production later reached much higher figures.



PLAN OF CONCRETE PLANT—U. S. Transit Mix Concrete Co., Elwood, Ill.

Even this is remarkable, however: assuming each truck to carry six cu. yd. of concrete, a run of 223 yd. per hour means a loaded truck through the plant every 95 seconds. This is a clear indication of what can be accomplished by careful management coupled with superior equipment engineered to fit the particular job it is called upon to do.

BUTLER BIN COMPANY - - WAUKESHA, WISCONSIN

CEMENT ACTIVATION

MAKES ALL THE CEMENT GO TO WORK

- Increases Flowability 100-200%

or

- Reduces Water 10-20%
- Increases Strength 15-20%
- Reduces Shrinkage, Absorption, Segregation

"Workability is a Measure of the Quality of Concrete"[1]

Workability, flowability, plasticity—the true flow of concrete, vitally concerns strength and durability. To achieve "place-ability" through the addition of more mixing water in order to get the concrete to where it is desired that it should harden—is doing so at a needless sacrifice of strength, density and excessive shrinkage.

Just as modern science today approves and permits the addition of grinding-aid admixtures to the clinker in manufacturing Portland cement,[2] so has the acceptance developed of the value of equally infinitesimal quantities of ingredients which promote cement activation at the time of use.

The latent powers of cement are a well known technological fact. To awaken or activate those powers has long been the goal of scientific research, in order that all of the cement may be put to work.

A minute quantity of harmless vitamins stimulates the human system—a tiny quantity of Horn's product stimulates the ever dormant powers of cement. Hence, the name—VITAMIX.

[1] J. D. Roberts J.A.C.I. 1931.

[2] P. H. Bates, President, American Concrete Institute —1935.

VitamiX

FOR CONCRETE
SPEED AND ECONOMY FOR DEFENSE

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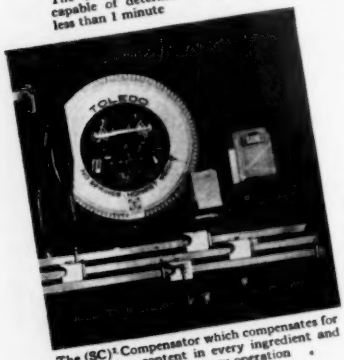

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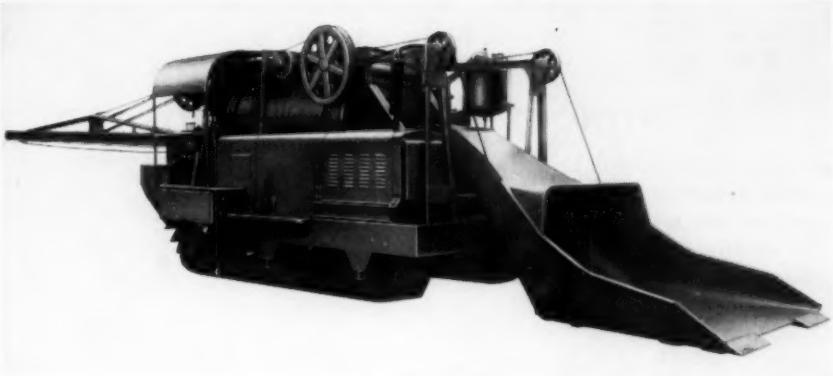

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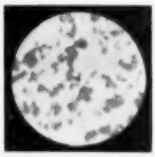
CEMENT DISPERSION

HOW CEMENT DISPERSION WORKS

Investigation shows that with 28 days curing only 50% of the cement hydrates. [Anderegg and Hubbell, A. S. T. M. 29 II 554 (1929)].

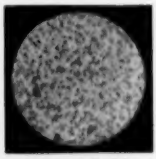
WITHOUT CEMENT DISPERSION

Cement particles in their normal state in water tend to gather in bunches; i.e., flocculate. Water never reaches some particles and many are only partly hydrated. This reduces the effectiveness of the cement, entraps water within the clumps, requires an excess of water for placement and often results in bleeding and segregation. See photomicrograph at right.



Cement suspended in water
UNDISPERSED

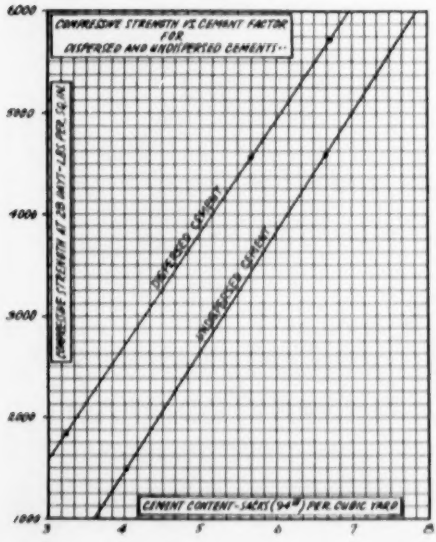
WITH CEMENT DISPERSION



Cement suspended in water
DISPERSED

The dispersion principle operates to drive each particle apart, thus exposing all the cement particles to the vital hydrating action. See photomicrograph at left.

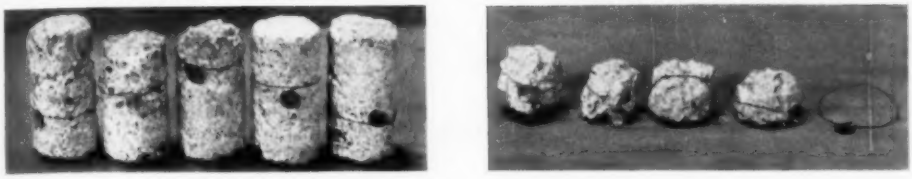
This dispersion makes the cement usable to its maximum efficiency since all the water is made available for lubrication of the mix and the entire surface area is exposed for hydration.



Dispersion with Cement by making the cement more effective produces a higher strength for any given cement content, or conversely, requires a lower cement content for a given strength, as shown in chart at left.

- - - - - ECONOMY

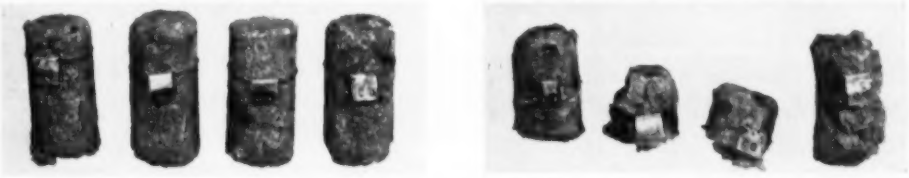
DISPERSED CEMENT **UNDISPERSED CEMENT**



After 150 Cycles of Freezing and Thawing

At left, Concrete with Dispersed Cement at equal consistency and identical in mix (except for 15% reduction in water) with the concrete using Undispersed Cement as shown at right. *Tests by Leading University.*

DISPERSED CEMENT **UNDISPERSED CEMENT**

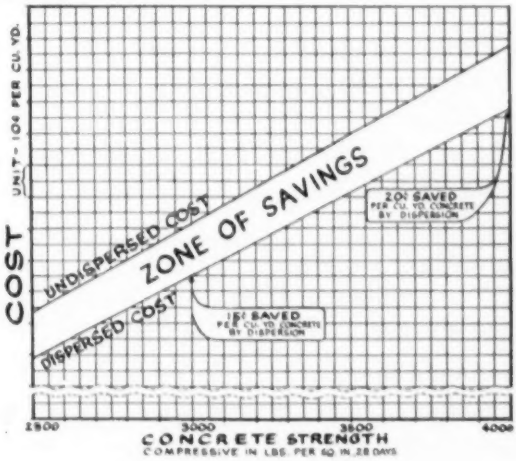


After two years exposure to 8% Magnesium Sulphate Solution (Renewed Weekly)

At left, Concrete with Dispersed Cement at equal consistency and identical in mix (except for reduction in water) with the concrete using Undispersed Cement at right. *Tests by Master Builders Research Laboratories.*

Whatever Your Concrete Requirements are — whether they be for low strength or high strength, for a 5-year shed or a 50-year Dam — **You Produce Them at Lower First Cost** with concrete designed employing **Cement Dispersion.** In addition, subsequent costs for **Maintenance** are lower.

1. **Concrete of Given Strength is produced at Lower Cost.**
2. **Stronger and More Durable Concrete is produced at a Given Cost.**



Based on cement at 50c per sack
Calculated from data on opposite page.

Send for Research Papers No. 35, "Application of the Principle of Cement Dispersion to Portland Cement" and No. 36, "Economics of Cement Dispersion".

THE MASTER BUILDERS COMPANY

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A part of PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE Vol. 38

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

Vol. 13 No. 5 7400 SECOND BOULEVARD, DETROIT, MICHIGAN April 1942

President's Address

By REAR ADMIRAL BEN MOREELL*

It is with the greatest regret that I have been forced to forego the pleasure and the distinction of addressing you in person and face to face on the occasion of this annual convention.

I am sure you all understand why it must be that my official duties now take precedence. The country is facing a great crisis—its greatest crisis. There is no shadow of doubt as to that. We are now entered on a struggle, on the outcome of which is balanced the hopes and ideals of ourselves and of all those who will come after us. No price is too great to pay for the preservation of those ideals. There must be no indecision in our willingness to pay that price.

We see ahead two roads. To the right, we see honor, idealism, peace. To the left, we see degradation and spiritual slavery. Hard work, suffering and deprivation line both roads. There is no choice there. There are things that are more precious than life itself and brave men do not hesitate when the choice is offered. We have made our choice.

What can an institution like ours contribute to this struggle? The whole is made up of the sum total of its parts. Therefore, each individual's first obligation is to perform his assigned task with all the devotion and loyalty and strength which lies within him.

Secondly, such groupings of loyal and patriotic citizens as our Institute must form the nuclei around which the bulwarks of resistance are erected. This will be a long and a hard war and as in all wars, there will be heard those timid souls who will cry out for peace under any condition, even

*Chief, Bureau of Yards and Docks, and Chief of Civil Engineers, United States Navy, by telephone from Washington, D. C., Thursday, February 19 to American Concrete Institute on the occasion of his retirement as its President.

of dishonor and servitude. They must be spurned. Those of us who see clearly must stand fast in the face of the enemy and in the face of our own weak-kneed and vociferous brothers. This organization, bonded with like organizations throughout the land, must and will assume this as their allotted task.

I send you my greetings and my wishes for a continuation of our past success, and may God give us strength to perform, each one, his sacred duty.

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Early Concrete Volume Changes and their Control*

By M. A. SWAYZE†
Member American Concrete Institute

SYNOPSIS

The types of volume changes occurring in concrete, in order of their incidence in a freshly mixed batch, are caused by water absorption, sedimentation, cement hydration, thermal change and wetting or drying of the concrete.

Cement hydration causes relatively large amounts of water to be absorbed by concrete in the first 24 hours. Where curing water is not supplied during the period when concrete is heating up from cement hydration, tests with Carlson strain meters have shown the possibility of offsetting thermal expansion by means of the autogenous shrinkage. At the time when the concrete reaches maximum temperature, the delayed curing leaves it in an unsaturated condition.

If water is supplied to concrete during the cooling period, the induced volume change from absorption decreases shrinkage from heat loss.

This treatment will materially decrease the large thermal shrinkages which produce cracking in concrete whose final structure is established at high temperatures. It is applicable to highway slabs and structural members of similar size, but will be less effective in large masses.

An appendix presents some of the significance of the paper for the highway engineer. It deals with the harmful effects on highway surfaces of too liberal application of curing water through curing mats at early ages.

The volume changes which occur in freshly mixed concrete have been given less consideration than those which take place after hardening has established a rigid structure. Yet all of these changes are of importance to the designer and user of concrete, since they may effect not only yield in terms of sacks per cubic yard, but other properties such as cracking tendency, compressive and bond strength, and stability of exposed surfaces. The purpose of this paper is first to discuss the various types of volume change, and then to point out how one type may be employed by the concrete engineer to control the permanent changes in volumes caused by another type.

*Received by the Institute Jan. 16, 1942.
†Director of Research, Lone Star Cement Corp., Hudson, N. Y.

The items contributing to volume changes in concrete, in about the order of their incidence in a freshly mixed batch, are: (1) cement solution and water adsorption, (2) sedimentation or "bleeding," (3) autogenous volume change, (4) thermal volume change, (5) induced volume change.

CEMENT SOLUTION AND ADSORPTION

When finely divided materials such as portland cement are mixed with water, an immediate small reduction in volume occurs. First, particles of the solid are dissolved by the water until it reaches the point of saturation, and in solution these dissolved materials occupy less total space than when in the solid condition. Second, when the surface of a solid particle becomes wet, the water molecules in direct contact with the surface become more densely packed than those at a distance from it. Both actions, therefore, reduce the volume of a cement paste and increase its density.

Since adsorption and solution are surface phenomena, slightly more shrinkage should be expected with the finer high early strength cements than with normal portlands. This is borne out by experiment, as shown in Table 1. All of the data are on neat mixes with 45.0 per cent added water (W/C 5.1 gal. per sack), with first observations on unit weight of pastes made 30 minutes after mixing. The figures on pastes are net, since air content was removed by vacuum. Cements used were a group of five normal portlands and the corresponding high early strength cements made at the same plants.

Note that the increases in paste specific gravity and decreases in yield from calculated values are roughly proportional to the surface areas; also that while the high early strength products generally have lower specific gravities their observed yields are practically identical with the heavier normal portlands. The data shown include some slight shrinkage due to chemical reaction of cement and water, and water adsorption by the gel formed.

Theoretically, some water in concrete is also adsorbed on aggregate surfaces. Practically, however, the total aggregate surface in concrete is so small in proportion to its bulk that this item may be ignored in

TABLE 1—EFFECT OF ADSORPTION AND SOLUTION ON PASTE VOLUME

	Normal Portlands	H.E.S. Portlands
Cements: average surface area cm.² per gm.	1730	2510
Cements: average specific gravity	3.16	3.12
Paste: calculated specific gravity	1.892	1.882
Paste: observed specific gravity	1.906	1.904
Yield, cu ft. per sack, calculated	1.156	1.163
Yield, cu. ft. per sack, observed	1.148	1.149

the design of concrete mixes. In general, correction factors for absorption of aggregates will include the adsorption.

SEDIMENTATION OR BLEEDING

The mechanics of sedimentation in concrete have been so thoroughly treated by T. C. Powers[1] that only the general effect on ultimate volume of concrete needs discussion here.

Following the placement of concrete mixes, settling of the solid portions and separation of clear water at the top surface begins immediately. This process continues until halted either by attainment of maximum compaction of the solids, particle interference, or the beginning of set. The quantity of water which collects on the upper concrete surface is dependent on the water-cement ratio, consistency, aggregate gradation, and also on cement fineness and other characteristics.

While there is no reduction in total volume of the system water-cement-aggregate by the phenomenon of bleeding, from the concrete designer's standpoint the amount of clear water which separates represents a very real shrinkage in his designed quantities. Whether or not the free water is removed, the top surface of the concrete will remain approximately at the level which existed when settlement ceased.

Loss of water by bleeding from plain concrete is unimportant, so long as bridging of aggregate particles is not serious. While cross-sectional areas are reduced, the net water-cement ratios are also lowered, and the density of concrete increased.

In reinforced concrete where the steel is restrained, losses in bond may become serious if bleeding is excessive, due to the bridging action of the steel forming water pockets on its under surface. Where the steel is not restrained, as in highway slabs, etc., the amount of bleeding is of much less import, as the steel remains in contact with the settling mass.

AUTOGENOUS VOLUME CHANGE

It is a recognized fact that, in the chemical reaction of portland cement and water the sum of the absolute volumes of dry cement and mixing water is greater than the volume of the hydrated product plus the remaining unused water[2]. Otherwise stated, the solid products of hydration have a higher specific gravity than the unhydrated cement minerals plus water for their hydration. Added to this condensation is an increase in adsorbed water. The fineness of the cement gel is much greater than that of cement, and more water is adsorbed, thereby increasing the magnitude of this volume change. Since both these changes are inherent within the cement-water reaction itself, they both come under the general classification of "autogenous" change.

(1) See "References" at end of paper.

Measurement of autogenous volume change may be done by several methods. The mixture with some excess water may be sealed in collapsible vessels, and changes in specific gravity noted; thimbles containing the paste may be placed in dilatometers with observation of the fall of the capillary water column[3][4]; or the mixture may be placed in a pycnometer or similar vessel and increases in weight determined, keeping the water level constant.

It must be kept strictly in mind that autogenous volume change as above defined and measured relates to the whole system-paste plus excess water. For this system the overall autogenous change is always shrinkage. The concrete engineer, on the other hand, is concerned only with the external dimensions of the solid component. Whether or not the hydration reactions result in shrinkage or expansion from these dimensions depends on the type of reaction of the cement minerals.

In the study of cement hydration made in the Lone Star Research Laboratory, Dr. L. S. Brown[5] observed that the tri- and dicalcium silicates hydrate by first going into solution, with a following precipitation of the finely granular hydration products on all surfaces wet by the solution—those of aggregate grains, cement particles, gypsum flakes, calcium hydroxide crystals and even air cells. This type of hydration he appropriately termed "accommodative", since the hydration products adapt themselves to the open space available.

Free lime and magnesia, on the other hand, hydrate in place, and their reaction with water is therefore "unaccommodative", since the solid hydrated grains occupy more space than did the grains of CaO or MgO before hydration. Tricalcium aluminate was found to have an accommodative hydration at ordinary concrete temperatures, but tended to hydrate in place at higher temperatures, such as in the autoclave test for soundness.

The microstructure of the hydrated lime silicate, the principal source of coherence and strength in concrete, is of prime importance in the consideration of autogenous volume change. Brown found this precipitate to be a gradually thickening crust, forming on all exposed surfaces. The preparations observed were cement pastes sealed in micro slides. The pastes were especially prepared so as to possess relatively high concentration of solid and at the same time preserve clear visibility. The water-cement ratios were about 1.5 to 2.0 by weight, which is very low for microscopic mounts. Clear visibility was obtained by levigation of cements and ground clinker with commercial ether to remove the ultra-fines. The cement grains used were closely sized at 15-20 microns.

Hydrolysis of the cement silicates in cements so prepared always resulted in the formation of this finely granular crust; rarely more than eight hours elapsed before its becoming visible. Under the conditions

of the tests, the diameters of individual granules at a maximum never were much above the limit of microscopic resolution (0.5 micron), and by varied manipulation it was possible to show that the earlier formed portion was well below this limit. It necessarily follows that pore spaces between the granules should have dimensions similar to the granules themselves, as was observed.

While neither granules nor pores have ever been made directly visible in a normally prepared, cured and hardened cement paste, Brown believed that his preparations reliably revealed the nature of the gel structure and gave a fair approximation of pore and particle sizes, presumably more nearly approaching the finer, earlier formed crusts. Other reports in the literature suggest pore spaces of 2-4 millimicrons in hardened cement pastes. In the light of the above observations, pore dimensions of 30-50 millimicrons in gels of ordinary concrete or of 10-20 millimicrons in high early strength concrete at the age of 12-14 hours are not hard to visualize.

Brown's observations are of further interest in that the size of directly visible particles in the gel crust varied with cement composition and other conditions. Cements with high A/F ratios yielded finer maximum sized grains than those with low A/F ratios. With either one, grain size was reduced by presence of gypsum as compared with its absence. Finer grain size resulted with curing in cool water, coarser grains forming in hot water.

These gel grains may be single solid particles, or they may consist of clumps of grains of much smaller dimensions. If the latter is the case, the internal pore spaces within the clumps are of a much smaller order than the external spaces between grains, and are far below the power of microscopic resolution. At all events, the surface area of the hydrated gel is much greater than that of unhydrated cement, and therefore, its power of adsorption is much greater.

There is little doubt that all of these observations are associated with autogenous shrinkage of interest to engineers. Several illustrations might be offered. Cement pastes sealed in glass vials with excess water generally show no shrinkage after hardening, but instead eventually expand enough to crack the glass. On the other hand, if excess water is small in amount, it is absorbed and following this the paste shrinks away from the glass. 6x12-in. concrete cylinders smoothly troweled flush with the top of the mold show subsidence of the upper surface as much as $\frac{1}{16}$ in. after bleeding water is reabsorbed: cylinders which have excess water continuously supplied show no such shrinkage. This difference is most apparent with rich mixtures.

The basic principle underlying this behavior is not entirely clear. There is a strong suggestion that surface tension of water may be an

important factor, with the tensile strength of water in capillaries also involved. Either of these items represents a force of considerable magnitude. Assuming the presence of such forces the writer offers the following explanation for the autogenous shrinkage of a solid concrete mass.

Pore spaces in the hydrated gel may be considered as ordinary capillary tubes or spaces between planes, with this difference: instead of the capillary walls being rigid, in the early hours of hydration they are free to move and to allow shrinkage to take place until the gel attains a strength sufficient to resist the capillary forces within. After this age the mass can no longer be considered as plastic.

The strength required to withstand these internal stresses tending to shrink the mass may be calculated by use of the formula for surface tension.[6]

$$T = \frac{ahdg}{2}, \text{ where}$$

T = surface tension, dynes per centimeter
a = radius of capillary or space between planes, cms.
h = height of rise of liquid in cms. ($h/70.4$ = pressure, psi.)
d = density of liquid (1.0 for water)
g = gravity, cgs units = 980

Assuming capillary sizes of 40 millimicrons for ordinary concrete and 15 millimicrons for high early strength concrete at the age of 12 hours, the values for h in terms of compressive strength would be 530 and 1410 psi. respectively. These calculated strengths are close in magnitude to observed strengths at this age.

To determine the potential early shrinkage possible from autogenous volume change, a series of tests was made, results of which are shown in Table 2. The same cements and pastes employed in tests for data in Table 1 were used. Calibrated 25 cc. pycnometers were half-filled with weighed quantities of cement slurry containing 45.0 per cent added water, air cells in the slurry removed by vacuum and agitation, and then the vessels were completely filled with distilled water. The temperature in all tests were maintained at 74.0 F. in a constant temperature bath. Weighings were started at the age of 30 minutes, and were made in a room 3 to 4 degrees colder than the bath to prevent overflow. The waterlevel in the pycnometers was adjusted to constant level just prior to each weighing.

The data show that water absorption began at a relatively rapid rate for all ten cements tested. At the age of 7 hours the absorption of the high early strength products was about 100 per cent higher than for the normal cements, while at 24 hours the excess was about 50 per cent. At later ages differences between the two types diminished.

TABLE 2—WATER ABSORBED BY CEMENT PASTES, 45 PER CENT WATER—
PER CENT ABSORPTION BY ABSOLUTE VOLUME OF CEMENTS

Cement specification	PLANT									
	A		B		C		D		Special Blend	
	Norm.	H.E.S.	Norm.	H.E.S.	Norm.	H.E.S.	Norm.	H.E.S.	Norm.	H.E.S.
Age of Paste	(Initial volume of paste + extra water determined at age of 30 minutes)									
1 hr	0.12	0.12	0.36	0.13	0.10	0.14	0.10	0.22	0.12	0.04
2 "	0.42	0.28	0.65	0.32	0.20	0.36	0.24	0.35	0.23	0.25
3 "	0.54	0.51	0.87	0.66	0.35	0.61	0.38	0.74	0.36	0.44
4 "	0.69	0.85	1.01	1.03	0.48	0.95	0.58	1.28	0.59	0.80
5 "	0.86	1.24	1.21	1.52	0.66	1.37	0.81	1.85	0.82	1.20
6 "	1.09	1.76	1.40	1.97	0.88	1.82	1.11	2.44	1.11	
7 "	1.33	2.37	1.62	2.46	1.11	2.32	1.42	2.97	1.37	2.33
14 "	3.65	5.70	3.67	5.40	3.06	5.50	3.97	5.64	4.67	6.09
24 "	5.26	8.40	5.27	7.36	5.07	8.10	5.12	7.07	5.52	7.72
2 days	7.31	10.53	7.72	9.22		10.60	7.12		7.95	10.64
3 "	8.80		9.82			11.94	8.48			
4 "	9.90		11.05		9.00	12.70	9.32	10.86	10.58	13.45
5 "	10.65	13.10	11.68	12.16	9.75	13.22	9.92	11.40	11.36	14.18
6 "	11.12	13.43	12.17	12.66	10.37	13.59	10.46	11.83	11.94	14.66
7 "	11.48	13.68	12.60	13.17	10.80	13.82	10.88	12.17	12.40	15.00
14 "	12.52	14.70	14.00	14.62	12.12	14.60	12.15	13.60	13.66	16.11
28 "	13.38	15.36	14.94	15.52	13.03	15.25	12.95	14.77	14.34	16.67

It should be noted that heat of hydration played no part in these tests, all of the pastes being held at a moderate and constant temperature. In normal concrete operations the higher early temperatures generally attained with H.E.S. cements would have materially increased their rates of absorption over those of the normal products.

It must not be inferred from these data that any reduction in overall volume of the hardened pastes occurred during the test period. That no appreciable expansion took place was evidenced by the fact that all of the pycnometers remained unbroken during the full test period. The absorptions found should be considered rather as the maximum *potential* shrinkage which could occur in the pastes if no excess water were present, and if the mass remained plastic and free from particle interference.

THERMAL AND INDUCED VOLUME CHANGES

Expansion with rise in temperature and contraction with its fall cannot be measured quantitatively in fresh concrete, on account of the other types of volume change which are going on simultaneously. However, by consideration of the thermal properties of the concrete components we may logically assume that the early coefficient of concrete volume change is at least as great and probably greater than after hardening is complete.

Sand and coarse aggregates of good quality undergo no chemical changes in the process of hardening, and their thermal coefficients are therefore constant. While no data exist on this property of dry cement, it is by volume a minor constituent of concrete, being exceeded materially by even the water volume. Since other lime silicate minerals have thermal coefficients of the same order as siliceous concrete, cement may be assumed to expand in a similar manner. While the volume change of water is generally not thought of in terms of linear expansion, its thermal coefficient is 6 or 7 times that of hardened concrete.

Considering all of these components, an assumption that the thermal coefficient of fresh concrete is equal to or greater than that of 28-day concrete seems justifiable.

Induced volume change is the result of change in the total water content of hardened concrete. With loss of water by evaporation, shrinkage occurs; with water gain in concrete up to the point of saturation, expansion results.

CONTROL OF THERMAL VOLUME CHANGES

In recent years the realization that high concrete temperatures during the hardening period have as much or more influence than drying shrinkage on the cracking of concrete has prompted the specification of low and moderate heat cements for structures where heat dissipation is a

problem. However, so far as the writer is aware, no attempt has ever been made to offset the expansion of concrete during heat rise or contraction on cooling by taking advantage of other controllable types of volume change, such as autogenous shrinkage and induced expansion.

The age at which hardening concrete reaches a stage where stresses set up by thermal and induced volume changes are not relieved by plastic flow is problematical. Volume changes prior to this age are of no consequence except as they may affect later changes. In lean relatively harsh concretes this age may be relatively short, due to particle interference with flow. In richer and more plastic mixes the period may be lengthened to the time when the compressive strength equals or exceeds the forces of deformation set up by autogenous shrinkage. That this time exceeds the 12-hour period for both normal and high early strength cements in a 6-sack mix is strongly indicated by a series of tests on early volume change made in our laboratory with the Carlson strain meters.[7]

These meters measure both temperature and strain by variations in electric resistance of two elastic-wire coils, one of which is unstressed, and the other wound under tension on spools attached to each end of the meter. Change in ratio of the two resistances indicates strain, while their resistance in series is a measure of temperature. The minimum measurable strain is about 4 millionths of an inch per inch. Sensitivity to strain is very high—a one pound load on one end of the meter causes a deformation of over 6 times the minimum measurable value. The coefficient of thermal expansion of the meters used was 7.5 millionths, which was identical with values determined at 28 days for concrete used in the tests.

In the volume change tests, results of which are shown in Table 3, and the volume changes plotted in Fig. 1 and 2, concrete slabs 36 x 36 x 10 in. thick were cast, using a mix containing 6.0 sacks of cement per cu. yd., and having a slump of 2-in. Aggregates were well graded up to 1½-in. maximum size. The strain meters were set horizontally at a depth of 5-in., and parallel with the side forms 6-in. in from the center of the faces. Readings for thermal expansion and strain were taken at frequent intervals for 15 hours and then daily. One normal portland and three samples of high early strength cement, all with relatively high C_3A contents, and all made at the same plant, were used in the tests.

In the first three tests shown the concrete surfaces were broomed at about 3 hours age, and at 4 hours were covered with damp burlap, then with dry burlap to retain the heat of hydration. The 1-in. wood form was sealed to prevent loss of mixing water. No excess water was available to the concrete surfaces up to the age of 12 hours. At this time an excess was added, so that the surface remained flooded. At 24 hours the

TABLE 3—EARLY VOLUME CHANGE OF 36 X 36 X 10 IN. CONCRETE SLABS

Concrete mix—1 : 1.82 : 3.43 BY VOLUME—2 in. Slump
All Data in Terms of Millionths Inches per Inch
(—) Indicates Shrinkage

Age of Concrete	TEST 1 Normal Cement H-1065				TEST 2 H.E.S. Cement H-1067				TEST 3 H.E.S. Cement H-1070				TEST 4 H.E.S. Cement H-1433			
	Tmp. °F	Ther. Exp.	Ind. Str.	True Str.	Tmp. °F	Ther. Exp.	Ind. Str.	True Str.	Tmp. °F	Ther. Exp.	Ind. Str.	True Str.	Tmp. °F	Ther. Exp.	Ind. Str.	True Str.
15 Min.	72.5	0	0	0	72.8	0	0	0	71.8	0	0	0	72.9	0	0	0
1 Hr.	72.3	Sedimentation			73.3	4	0	4	72.0	2	0	2	74.2	10	−7	3
2 Hrs.	72.0	Changes Only			73.4	5	11	16	72.1	2	0	2	75.3	18	−15	3
3 "	71.8	0	0	0	73.6	6	15	21	72.3	4	11	15	77.4	34	−19	15
4 "	72.0	2	−4	−2	74.5	13	15	28	73.1	11	11	22	81.2	62	−27	35
5 "	73.1	10	−8	2	76.0	24	22	46	74.8	22	19	41	85.0	91	−23	68
6 "	74.6	21	−23	−2	79.0	46	−7	39	77.4	42	23	65	90.7	133	−23	110
7 "	76.6	36	−42	−6	82.7	74	−56	18	81.1	70	−4	66	97.2	182	−27	155
8 "	78.0	46	−53	−7	87.1	107	−90	17	86.6	111	−72	39	101.0	211	−30	181
9 "	—	—	—	—	88.6	118	−101	17	91.8	150	−110	40	104.2	234	−38	196
10 "	84.0	92	−87	5	92.3	146	−116	30					106.5	252	−38	214
11 "	—	—	—	—					97.2	190	−137	53	107.9	262	−38	224
12 "	90.5	140	−110	30	97.9	188	−128	60	99.5	208	−144	64	108.9	270	−38	232
13 "	—	—	—	—	100.2	206	−131	75					109.7	276	−34	242
14 "	91.7	149	−110	39					101.4	222	−148	74	110.3	280	−34	246
15 "	—	—	—	—	101.7	216	−135	81	101.4	222	−144	78	110.5	282	−34	248
24 Hrs.	92.0	151	−91	60	101.2	213	−128	85	97.7	194	−125	69	101.8	217	−15	202
2 Days	*	0	11	11	*	0	0	0	*	0	19	19	76.6	28	11	39
3 "	*	0	11	11	*	0	11	11	*	0	23	23	73.0	1	15	16
4 "	*	0	11	11	*	0	11	11	*	0	—	—	—	—	—	—
5 "	*	0	11	11	*	0	15	15	*	0	11	11	—	—	—	—
6 "	*	0	11	11	*	0	15	15	*	0	11	11	*	0	11	11
7 "	*	0	11	11	*	0	15	15	*	0	11	11	*	0	11	11
Max. Temp. Rise	20.2 F.				28.4 F.				29.6 F.				37.6 F.			

Underscoring indicates age at which surface was flooded.

*Corrected to original temperature of fresh concrete.

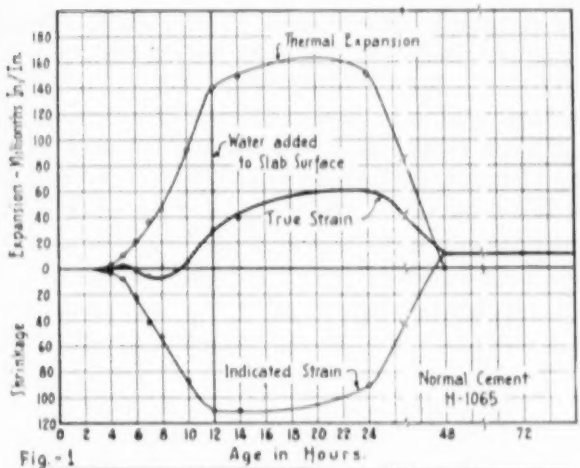

Fig. 1 — Early volume changes of concrete slab showing operation of Carlson meter

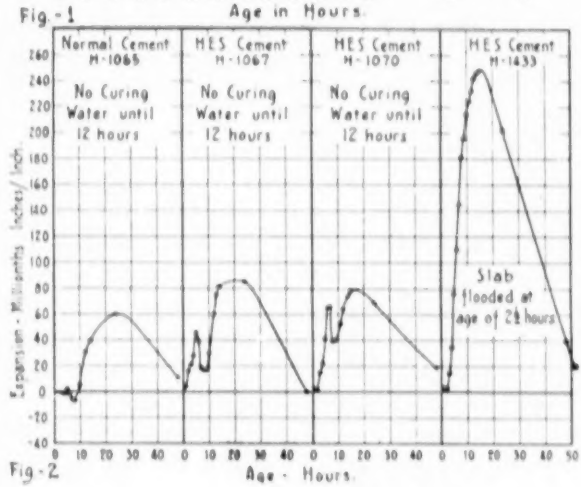

Fig. 2 — Early volume changes of concrete slabs

forms were removed and the surface kept covered with wet sand for 7 days.

In the fourth test (H1433) the surface was provided with a small dike, and was kept flooded with water after the age of 2½ hours. A cellophane and dry burlap cover was supported just above the water level to retain the heat. This provided slightly better heat insulation, and the concrete reached a higher maximum temperature. In all four tests the forms were loosened soon after the concrete could support its own weight, to avoid possible negative stresses set up by restraint at the sides.

It is apparent in the first three tests that while the concrete was expanding due to thermal changes, shrinkage due to absorption and combination of the mixing water was also taking place. The columns headed

"Thermal Expansion" under each cement show the expansion which took place in the meters due to heat rise. The data for the first cement are plotted in Fig. 1.

Since we have previously seen that fresh concrete should have a thermal expansion equal to or greater than the meters, the expansion of the concrete from heat alone should have at least equalled the values shown. The fact that all of the meters in the first three tests were under increasing negative strains after the first few hours indicates definitely that the meters tried to expand more than the concretes.

The third columns for "Indicated Strain" show the differences between the actual concrete expansion and the corresponding free thermal expansion of the meters, except for a possible correction for water film around the meter ends. Consideration of the data will show, however, that if such a film originally existed, then the negative values shown for "Indicated Strain" are too small. The "True Strain" columns shown in the table and plotted in Fig. 2, therefore, show the maximum possible expansion of the concretes with rise in temperature. Precautions were taken against water film by enlarging the meter ends, and by avoiding any possible disturbance of their position in the fresh concrete.

It is highly significant that where evaporation was prevented but additional curing water withheld for 12 hours, as in the first three tests, the overall expansion during the heating period was small, and of the same order in all three concretes. In the fourth test the concrete had all the curing water it could absorb, showed little autogenous shrinkage during the heating period, and the overall expansion was of a high order. It is also significant that very soon after the surfaces were covered with water, autogenous shrinkage essentially ceased in all four cases.

So far the changes in volume found to have taken place are of little interest to the concrete engineer, except as they may influence changes to follow. During at least the early part of the heating cycles the concretes were essentially plastic, and even up to the 12 hour age were responding to control by adding or denying curing water to the surface. However, if at some age at or about the time when maximum temperatures were reached the "final" structure of the concrete was established, then subsequent volume changes are of great importance, as is the ability to control those changes.

During the cooling periods, between the ages when maximum temperatures were reached and the time when the concretes were down to room temperature, the specimens again showed a different behavior. In the first three cases, water had evidently been withdrawn from the larger pore spaces by absorption, and the specimens were presumably in an unsaturated condition. Therefore, during the cooling period, while they were shrinking from loss of heat, the decreasing negative strains

shown by the meters indicate that they were also expanding from the force of induced volume change. In the fourth test there was no evidence of such a compensating force, the concrete being apparently saturated with water when it reached its maximum temperature. Consequently no offset for the loss of heat was possible, and the net shrinkage was 3 to 4 times greater than in previous tests. While some of this difference was due to the higher temperatures found in Test 4, correction for temperature differences would still leave the cooling shrinkage 2½ to 3½ times as much.

CONCLUSIONS

Consideration of all the foregoing leads to the conclusion that some considerable control of early volume changes in concrete is in the hands of the concrete engineer. By preventing evaporation from the fresh concrete surface after water from bleeding has been removed, but withholding curing water until the concrete is nearing its maximum temperature, he can then decrease the later thermal shrinkage by effective use of curing water during the cooling period.

The thickness of concrete members in which this curing control of thermal volume change will be effective seems to be a matter connected with the properties of capillaries, with rate of water flow through them as the determining factor. That the average highway slab and similar structural members will respond to this control, there seems little doubt. While mass concrete should be less quickly responsive, there are some mitigating circumstances in such cases. In general, cooling is slower, giving curing water more time to penetrate the larger mass. Second, it has been indicated that the silicate gel grains of moderate and low heat cements may be larger than with cements of higher A/F ratio. If this is the case, the pore spaces between grains should also be larger, and less opposition to water flow through the capillaries will be encountered.

Acknowledgment is hereby made to Dr. L. S. Brown, Research Petrographer of the Portland Cement Association, for cooperation in preparing the statements concerning his work on the micro study of cement hydration.

REFERENCES

1. T. C. Powers: "The Bleeding of Portland Cement Paste, Mortar and Concrete." P.C.A. Research Laboratory Bulletin 2. Condensed version, ACI JOURNAL June 1939, *Proceedings* V. 35, p. 465.
2. Lea & Desch: "The Chemistry of Cement and Concrete." p. 175.
3. W. C. L. Hemeon: *Ind. Eng. Chem. 27*, 694, (1935).
4. T. C. Powers: *Ind. Eng. Chem. 27*, 790, (1935).
5. L. S. Brown: Data from an unpublished paper given at A.I.M.M.E. meeting at Tulsa, Okla., Oct. 26th, 1940.

6. Herbert Fruendlich: "Colloid and Capillary Chemistry".
7. R. W. Carlson: "Five Years Improvement of the Electric-wire Strain Meter." *Eng. News Record*, May 16, 1935.

APPENDIX—REPORT OF CONCRETE VOLUME CHANGE

In the body of this report on concrete volume change it has been seen that fresh concrete will absorb rather large quantities of curing water, beginning as soon as the surface water resulting from bleeding has been removed or reabsorbed. While the main purpose of the report was to present a means for controlling the early volume changes, the influence of curing control on the quality of the concrete surface should not be overlooked. In the following discussion the type of concrete member under consideration will be highway slabs. However, the remarks will be equally applicable to other similar types of construction.

In his paper on the bleeding of concrete, T. C. Powers has shown that a highway slab, for example, at the end of the bleeding period will have a zone of maximum compaction at the bottom, a transition zone of variable water content above it, and near the top surface a possible third zone of constant water content and constant rate of settling up to the time of start of set. The water content of the concrete will be lowest in the base zone, and increase until it is at a maximum at the top surface. This condition is exactly the opposite from the one desirable from the standpoint of surface resistance to frost action.

Finishing of the slab—surface-brooming, and troweling of edges—must follow the cessation of bleeding rather quickly, or the concrete will have lost its workability.

If the application of curing water is started very soon after the finishing operation, absorption of this water will tend to maintain any inequality in water content existing between the top and bottom of the slab, by halting autogenous shrinkage. On the other hand, if use of curing water is delayed, autogenous shrinkage forces will act to redistribute the water in the concrete, and also to compact the less dense structure near the top of the slab.

During the delay in curing, evaporation of water from the surface must be prevented, or rapid losses of water will cause development of surface shrinkage cracks. Damp burlap or other curing mats should therefore be applied, but should contain no free water which can be absorbed by the concrete until maximum compaction of the surface has been obtained. Application of bituminous surface membranes serve this purpose very well if applied in sufficient thickness, but have the disadvantage that they later prevent any ingress of curing water.

While delayed curing will materially improve the surface condition of concrete by increasing its density, a still better reason exists for this delay when burlap or similar covering is employed. All burlap, even of the best quality, contains some organic materials soluble in water which delay or may even inhibit the reaction between the cement and water. To furnish an example of this action, a sample of new burlap was soaked in water for 12 hours, then wrung out and the water used in a batch of concrete. The compressive strength of this concrete was reduced to about 85 per cent of that obtained with clean water. In this test the soluble materials from 4 lbs. of dry burlap were dispersed in about 40 lbs. of water. If the amount of burlap used had been greater, or the ratio of water to burlap had been less, a greater reduction in strength would have resulted.

The presence of mildew or other molds in burlap or similar materials greatly increases both the amount and effect of the soluble organic material. Since the distribution of

mold spores is almost universal, the condition of any sort of curing blankets made from vegetable fiber must be under suspicion if they have remained in a damp state for any material length of time. The presence of mildew or molds is easily detected by their characteristic musty odor.

Under such circumstances delay in the addition of curing water is imperative. Otherwise the water will dissolve organic material from the curing blankets and carry it directly into the top surface of the concrete, where all further hardening may be inhibited or at least greatly delayed.

Fig. 3, 4 and 5 illustrate the effects of time of applying curing water on the stability of a road surface. In the area shown in Fig. 3 new burlap blankets had been laid on the freshly finished concrete and liberally soaked with water beginning 2½ to 3 hours after the concrete was placed. The surface condition in the area shown by this picture was aggravated by the fact that it was at the bottom of two slight grades on either side, so that water leaching through the burlap concentrated in the low area. On removal of the covering it was noted that the concrete surface was covered with a brown stain, and that very little hardening of the surface had taken place in the stained areas. The edges and unstained portions of the slabs had a normal hardness.

Following discovery of this condition, further concreting was delayed until the burlap had been laundered. When work was resumed with the cleaned blankets, the early application of curing water was continued, with results as shown in Fig. 4. Brown stains on the surface were still present, and surface set in these areas was very slow, although the depth of affected spots was less. This improvement in surface condition may be attributed to loss of some of the soluble material in laundering, or to the observed runoff of much of the excess curing water at the edges of the slabs.

Later the curing process was changed. Covering with the blankets was held up until further delay would have caused development of cracks from surface shrinkage. When laid, the burlap received only enough water for the first 24 hours to keep it damp and thereby prevent loss of water from the concrete. Real curing began after the burlap had been removed, and the concrete had been covered liberally with roadside earth. Fig. 5 is typical of the remainder of the road after the curing methods were revised.

All of these pictures were taken 18 months after the pavement was laid, but represent almost exactly the conditions as they existed when the road was opened to traffic. In the writer's opinion, the history of many unexplained cases of highway scaling may be similar to the case cited above.

In view of the facts presented in this appendix, as well as the data in the foregoing report, the writer recommends that serious consideration by highway engineers be given to the matter of curing control in their concrete specifications.

All normal concrete contains water considerably in excess of that required for complete cement hydration. Therefore, in the early stages of curing, the important thing is not to provide additional water, but to prevent excessive losses from the surface which would result in development of hair-checking, shrinkage cracks or a chalky finish. Later, after the slab has reached its maximum temperature and is cooling, addition of plenty of curing water will lessen the chance of structural cracking, provided the slab is not already saturated.

With these concepts, curing blankets of any material should be considered purely as a means of preventing evaporation, instead of a conveyor of new water to the surface. The time of application of curing blankets must depend on temperature and humidity conditions during the paving operation. In cool, damp weather the operation can be

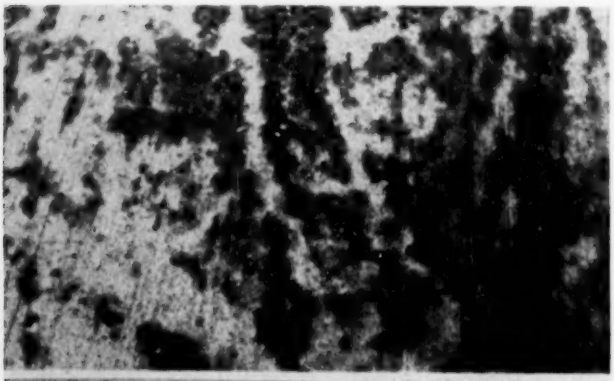

Fig. 3—Scaled concrete surface caused by application of new burlap, heavily soaked with water, 2½ hours after placing concrete

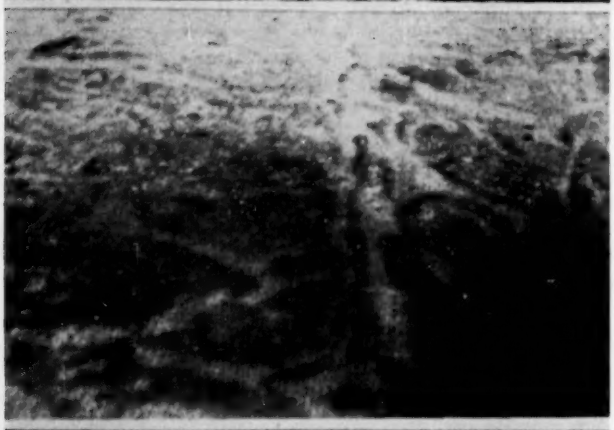

Fig. 4—Concrete surface scaled from application of laundered burlap heavily soaked with water, applied 2½ hours after placing concrete

Fig. 5—Concrete surface cured under laundered burlap applied late, kept moist only

greatly delayed: in hot or dry climates covering may have to follow the finishing operation closely. However, only enough water should be supplied to keep the covering moist—an excess can have no beneficial effect on the surface condition during the first hours of the slab's life.

Discussion of this paper should reach the ACI Secretary in triplicate by July 15, 1942, for publication in the JOURNAL Supplement.

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Progress in the Long-Time Study of Cement Performance in Concrete*

By F. R. McMILLAN†
Member American Concrete Institute

The incentives for the "Long-Time Study of Cement Performance in Concrete," announced by the Portland Cement Association in the *News Letter*, ACI JOURNAL, January 1941, are found in a paragraph from that announcement as follows:

"It is of concern alike to the producer and consumer of portland cement that the performance of concrete under the rigors of service cannot always be predicted or relied upon with certainty. These predictions are based upon premises which depend upon an attempted correlation of laboratory research and observation of field operations with field performance. Since the non-reliability of the predictions seems to be due to a lack of proper correlation between laboratory and field observations and field performance, the Portland Cement Association is about to undertake a long-time study of the performance of cement in concrete."

Announcement was made in the ACI JOURNAL for January, 1941 of the Long-Time Study of Cement Performance in Concrete now under way. This extensive investigation, which grew out of a suggestion made by P. H. Bates, is being carried out in accordance with the program prepared by an Advisory Committee of which Mr. Bates is chairman and consisting of eight representatives from outside the cement industry and four representing the cement industry.‡

The objectives in this investigation are twofold: First, to learn to what extent differences in cement affect the performance of concrete;

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†Director of Research, Portland Cement Association, Chicago.

‡*From Outside the Cement Industry:* Chairman—P. H. Bates, Chief, Clay and Silicate Products Division, National Bureau of Standards, Washington, D. C.; Bryam W. Steele, Office of Chief of Engineers, U. S. Army, Washington, D. C.; Robert F. Blanks, Senior Engineer, U. S. Bureau of Reclamation, Customhouse, Denver, Colo.; P. J. Freeman, Principal Materials Engineer, Tennessee Valley Authority, Knoxville, Tenn.; T. E. Stanton, Materials and Research Engineer, California Division of Highways, Sacramento, Calif.; Roy W. Carlson, Massachusetts Institute of Technology, Cambridge, Mass.; F. H. Jackson, Senior Engineer of Tests, Public Roads Administration, Washington, D. C.; R. B. Young, Testing Engineer, Hydro-Electric Power Commission, Toronto, Canada. *From the Cement Industry:* R. G. Uhlig, Vice President and Operating Manager, Missouri Portland Cement Co., St. Louis, Mo.; Hubert Woods, Chemical Engineer, Riverside Cement Co., Riverside Calif.; Roy N. Young, Chemical Engineer, Lehigh Portland Cement Co., Allentown, Pa.; F. R. McMillan, Director of Research, Portland Cement Association, Chicago, Ill.; Ex officio: Frank T. Sheets, President, Portland Cement Association, Chicago, Ill.

second, to learn what factors are responsible where differences in performance are found.

The progress to date can best be presented in terms of the following three separate phases into which the program is naturally divided:

1. Collection of cements
2. The building of structures
3. Laboratory investigations

One of the requirements set up by the Advisory Committee for the first phase of this study was that there should be obtained a complete history of each cement during the progress of manufacture.

In the belief that the best record of the history of a cement during manufacture would be found in the product itself, special emphasis was placed on the collection of samples. Grab samples of the raw materials, the clinker, and the finished cement were taken at frequent intervals, usually every half hour. Where several kilns were used in making the test cements, the sampling of each was carried out separately. These grab samples were preserved in separate containers so that any desired study can be made of the uniformity of operations, or of the variations in the product. The usual mill control tests, petrographic studies of the clinkers, and many special tests were made, or are now in progress, all designed to give the fullest possible information concerning the test cements.

One hundred barrels of each of the 27 cements and a good supply of each clinker are available in sealed drums for use in the extensive laboratory studies comprising the third phase of the program.

As a further record of the uniformity of operation, burning temperatures were taken in some or all of the kilns used in producing the test cement at each plant.

Such other records were made during the preparation of the cements as seemed to be significant, so that the fullest possible data will be available for interpretation of any differences that may develop in the behavior of the cements.

The collection of these cements was started in December, 1940, and was completed in June, 1941. In all, approximately 1000 man-days were required of the field party in this part of the program. More than 24,000 individual containers were required for the samples collected.

The *second* phase of this study, which is now well advanced, is the building of the structures. The purpose is to have a series of structures of different types and in different localities built in such a manner that, in each series, the different cements will be used under conditions as nearly identical as is possible to make them in the field.

The three big projects are test roads built with the cooperation of the Public Roads Administration and the highway departments of three states. The first test road was started on July 31 and completed on August 8, 1941 in Audrain County, Missouri. In this road, 24 cements were used in rotation in sections 75 ft. long. Each cement was used five times, making a total of 120 sections of 22-ft. roadway.

During construction we had a large field party on the job, making moisture determinations, slump tests, and unit weights, checking the water going into the mix, making temperature measurements of the slabs and detailed observations on the bleeding and finishing characteristics of the cements and other significant features of the operations.

On the second test road, near Greenville, South Carolina, (finished on November 5) the same 24 cements were used. Here, however, each cement was used only four times, but the sections were 90 ft. long.

The test road in New York, originally scheduled to be first, had to be postponed until 1942 owing to a number of unforeseen difficulties. On this project 27 cements will be used, each four times, in single-lane construction. There will also be included in this project some extra features which will involve the further use of 18 of the 27 cements, making in all 180 test sections of single-lane road.

Second in importance to the three roads are two experimental farms in which all of the test cements are to be used in a number of oversized laboratory specimens built in the field and exposed to normal weathering. One of these farms is located near Chicago and the other in Perry, Georgia. The specimens on these farms are of three types: slabs 2½ by 3½ ft. by 6 in. thick, cast in place; hollow box-like specimens about 2½-ft. (cubes) in outside dimensions; short columns or posts 8 by 8 by 60-in. set endwise 2 ft. in the ground. Fig. 1 is a view of some of the specimens at the Illinois Farm.

The slab specimens are intended to simulate the exposure in a road slab. The boxes, which are to be kept filled with moist earth, simulate the exposure of concrete members extending from the ground line into the air.

For the slab specimens at each farm, there are three different types of aggregate, each used in three different mixes, making nine rows. For the box and column specimens there are two different types of aggregate, each used in six different mixes. The details of the aggregates and mixes used are shown in Tables 1 and 2.

Thus, for each of the 27 cements there are at each farm nine slabs, twelve boxes, and twelve columns. This gives 33 separate comparisons for the 27 cements at each farm, or a total of 66, not counting a number of additional series which have been added. When to this are added the

13 comparisons offered by the three major test roads, it can be seen that, admitting some non-uniformity in control, it is extremely improbable that any one cement will be more favorably treated than the others. If differences in performance develop which show that certain cements are superior or inferior in resistance, considering all of the structures as a whole, it will be a fair conclusion that some property or peculiarity of

TABLE 1—AGGREGATES

	Sand	Coarse Aggregate	Size
		Illinois Farm	
Slabs			
A	Missouri	Missouri Crushed Limestone	No. 4-2 in.
B	Springville*	Plainfield Gravel	"
C	Irving*	Hornell Crushed Gravel*	"
Boxes and Columns			No. 4-1 in.
D	Plainfield	Plainfield Gravel	"
E	Springville*	Plainfield Gravel	"
		Georgia Farm	
Slabs			
F	S.C. test road	Beverly Crushed Stone	No. 4-2 in.
G	Gaillard	Rockmart Crushed Limestone	"
H	Gaillard	Montgomery Gravel	"
Boxes and Columns			
I	Gaillard	Montgomery Gravel	No. 4-1 in.
J	Gaillard	Rockmart Crushed Limestone	"

*New York

TABLE 2—CONCRETE MIXES

Illinois and Georgia Farms

Mix No.	Cement Sk./cu. yd.	Slump in.	Method of Placing
Slabs			
1	5	3	Hand
2	6½	3	Hand
3	6½	8	Hand
Boxes and Columns			
1	6	3	Hand
2	6	1½	Vibrated
3	6	8	Hand
4	4½	3	Hand
5	4½	1½	Vibrated
6	4½	8	Hand

the cement has affected the concrete-making qualities. What particular factors are responsible for the differences must be sought in the data from the laboratory tests and plant observations.

Of importance also in the series of field structures is the project designed to study the effect of exposure of reinforced concrete to sea water in northern and southern latitudes and to fresh water in the north.

One hundred thirty-two concrete piles, 12 by 12 in. in cross-section, were made at Franklin Park, Illinois, with the Deitrick-Bullen Construction Co. as contractors, using 22 of the test cements. There were six piles for each cement, three were 30 ft. long and three were 20 ft. long. Fig. 2 and 3 show the making of these piles at Franklin Park.

The 30-ft. piles were shipped to Buzzards Bay where they were driven by jetting in the Cape Cod Ship Canal. The 20-ft. piles were shipped to Kingston, New York, where they are available for driving this coming spring in the Hudson River. The mixes and consistencies were:

1. 5 sk. per cu. yd. cement, 2-in. slump, vibrated.
2. 7 sk. per cu. yd. cement, 2-in. slump, vibrated.
3. 7 sk. per cu. yd. cement, 8-in. slump, hand-placed.
   (Local sand and gravel were used).

Sixty-six similar piles, 30 ft. long, using the same 22 cements, were made at Jacksonville, Florida, by the Ingram Construction Co., contractors. The driving of these piles in Salt Run, near St. Augustine, Florida, was completed on December 22.

While the piles were being made at Franklin Park, two 6 by 6 by 30-in. beam specimens were made for each mix and consistency for each cement. These 132 beam specimens were transported by truck to Eastport, Maine, where through the courtesy of C. E. Wuerpel, half of them are now on exposure alongside of his special test specimens and the other half are in storage for later exposure at the same place.

Mr. Wuerpel took readings with the sonic apparatus before the installation of these specimens and has made subsequent readings at each period of observation on his own specimens.

During the coming year, an additional group of piles will be placed at some sea water exposure in southern California. A farm will also be established at some point in the west for exposure of specimens to alkali soils.

The test cements are also to be used in thin sections of concrete like parapet walls on two or possibly more important structures where they will be exposed to severe changes in weather.

Of the third phase of this program—laboratory tests of clinker and cements—only brief mention will be made. The program requires that every type of laboratory test now recognized or later to be devised which will relate the characteristics of the cement to its performance under service conditions shall be carried out. This part of the work is well

Fig. 1 (above)—Specimens at Illinois farm

Fig. 2 (above) and 3 (at left)—Making concrete piles at Franklin Park, Ill.

under way and many data on chemical and physical properties of the cement and clinker are available. Petrographic studies of the clinker are also well advanced. In all of this work individual grab samples are being studied as well as composites.

The program as a whole is now more than half completed.

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Canal Lining Cured by Sprayed Coats of White-Pigmented Compound*

By O. G. BODEN†

SYNOPSIS

The expense and inconvenience of water curing by sprinkling 24 hours a day on a project where concrete canal lining was progressing as much as 1000 linear feet per day, were avoided by using sprayed curing compounds. An 8½ mile section of canal was coated with a clear compound plus burlap shading for three days. On the next section tests showed that a white-pigmented compound eliminated the need of burlap shading. Heat reflecting characteristics of the white-pigmented compound kept concrete temperatures as low as when the clear coated concrete was shaded.

Adequate curing of the extensive surfaces of concrete canal lining has been a major problem in construction of the Contra Costa Canal. Specifications originally required:

> Exposed surfaces of concrete shall be protected from the direct rays of the sun for at least three days. All concrete shall be kept continuously moist for at least two weeks after the concrete has been placed. . . In lieu of the above requirements, the contractor may, with the consent of the contracting officer, cure the exposed surfaces of the concrete by the application of an approved clear curing compound . .

Under normal operations, as much as 1,000 linear feet of canal has been lined in one day, and it is evident that water curing by sprinkling 24 hours a day for two weeks would be almost prohibitive from the standpoint of required equipment and the expense. The accumulation of water in the bottom of the canal would also greatly hamper other operations.

The contractor for the first section of approximately 8½ miles of lined canal elected to cure the concrete lining with clear curing compound plus shading for three days. Burlap was stretched across the top of the

*Received by the Institute Apr. 3, 1941—one of three papers by Mr. Boden about construction practices on the Contra Costa Canal, Delta Division, Central Valley Project. See "Machine Methods for Trimming Subgrade and Placing Concrete Canal Lining," ACI JOURNAL, Nov. 1, 1941; "Transit Mixers Used Effectively on Contra Costa Canal," ACI JOURNAL, January 1942—both previous papers also in this *Proceedings* Volume (38), p. 117 and 269 respectively.

†Construction Engineer, Bureau of Reclamation.

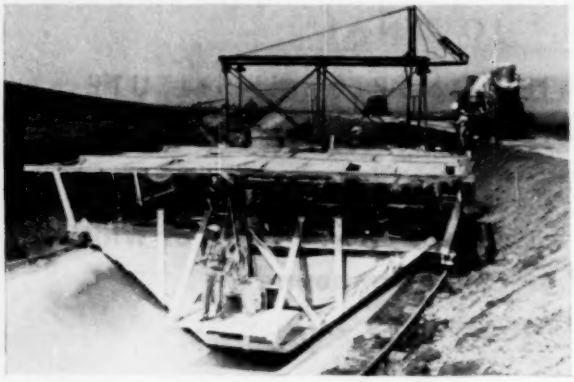

Fig. 1—The first coat of curing compound is applied from the rear of the traveling platform used by trowelers. The spraying equipment consisted of a supply tank and pressure pump driven by a small gasoline engine

canal prism and staked to the ground. This proved effective, except that during strong winds it was very difficult to keep the burlap in place. The contractor on the next section of about the same length elected to use the same curing methods. During the construction of this section, experiments were conducted to determine the effectiveness of a white-pigmented curing compound which would reflect the rays of the sun and, it was hoped, eliminate the necessity of shading the concrete. A number of recording thermometers were embedded in test sections of concrete lining coated with the white-pigmented compound and exposed to the sun. Others were placed in test sections of concrete lining coated with clear curing compound and shaded from the sun for three days. Results indicated that temperatures under white-pigmented curing compound were as low and sometimes lower than those which prevailed under the portions cured with a clear compound and shaded from the sun with burlap. In addition, the white surface continued to function in reducing temperature fluctuations for an extended period. Conclusions drawn from these and other investigations by the Denver laboratory resulted in the approval of the white-pigmented compound for use on the next section of canal lining.

Different kinds of white-pigmented curing compounds have been used, in both 2-coat and 3-coat applications, with excellent results. The first coat of compound was applied by spraying immediately after the final trowelling of the surface. The spraying equipment consisted of a supply tank and pressure pump driven by a small gasoline engine, all of which were mounted on the rear end of the travelling platform used by the finishers for final trowelling of the concrete surface. A compressor and pressure tank mounted on a truck were used in applying the succeeding coats.

Tests indicate that a coverage of 100 sq. ft. per gal. of white-pigmented curing compound containing 30 per cent non-volatile solids in the vehicle,

applied in 3-coats, or a coverage of 150 sq. ft. per gal. of white-pigmented curing compound containing 45 per cent non-volatile solids in the vehicle, applied in 2-coats, were practically equivalent to 14 days water curing. An added advantage of the curing compound is that the membrane remains much longer than the period usually required for water curing and so continues to function, to some extent, in retaining moisture and reducing temperature fluctuations due to the rays of the sun.

Specifications for that section of canal for which a contract was recently awarded contain the following provision:

the contractor may, with the consent of the contracting officer, cure the exposed surfaces of concrete by the application of an approved clear or white-pigmented curing compound: *Provided*, ... that protection from the direct rays of the sun will not be required if white-pigmented curing compound is used.

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Earthquake Stresses in Frame Structures*

By ROBERT E. GLOVER†
Member American Concrete Institute

SYNOPSIS

A way of utilizing the instrumental records obtained from past quakes to evaluate the earthquake resistant qualities of proposed or existing structures is developed. The effect of damping may be included and a method of using a torsion pendulum to obtain certain important results is described. The condition under which an earthquake allowance in the form of a transverse load proportional to the weight may be safely used is identified.

INTRODUCTION

The methods outlined in the following paragraphs have been found to be effective in extending the theory of vibration of beams and adapting the modification to the evaluation of stresses in certain types of structures when their supports are shaken by an earthquake. Since good accounts of the theory of vibration of beams with fixed supports may be found in the literature[1] [2] it will only be necessary, before taking up the new development, to review those parts which are useful for the present purpose. These are the following: The statement of Newton's law of motion for an element of the beam leads to a partial differential equation of the fourth order whose dependent variable is the displacement of the element from the position of no stress and whose independent variables are distance, measured along the centerline of the beam, and time. A solution to this equation can be found in the form of a product of a function of the length and a simple harmonic function of time. The substitution of a trial solution of this form into the partial differential equation will yield an ordinary differential equation of the fourth order from which the appropriate expression for the function of length may be determined. This solution will contain four arbitrary constants which must be determined by imposing the appropriate conditions at the sup-

*Received by the Institute Feb. 1, 1941.
†Senior Engineer, Bureau of Reclamation, Denver.
(1)(2)See "References" at end of paper.

ports. A peculiar situation arises when this is attempted since the right hand members of the four equations of condition are all zero. This prevents a solution of the simultaneous equations in the usual manner. In order to obtain a solution it is necessary to make the determinant of the system equal to zero. This can be accomplished by adjusting the modulus of the time variable whose magnitude has not, as yet, been determined. It is found that there is a succession of discrete values which will satisfy the requirements. With these values it becomes possible to compute three of the unknown constants in terms of the fourth. Each of these sets of constants will define a deflection pattern whose frequency of vibration can be computed from the discrete value of the time modulus associated with it. These patterns are referred to as "Modes." The meaning of these results, in physical terms, is that a beam can vibrate in several ways or "modes," and that each of these "modes" possesses its own characteristic frequency. This behavior is in sharp contrast to that of an idealized system, consisting of a concentrated mass held in position by a weightless spring, which has only one frequency of vibration. The difference in behavior can be traced to the fact that the masses and restoring forces in a beam are distributed whereas in the idealized system they are not. It remains to determine the magnitude of the fourth constant and thereby to find the amplitude of vibration. For a beam which is set into vibration by being pulled aside and released, the magnitude of the remaining constants are to be determined by fitting the initial curve of the centerline by a series of the mode functions in a manner very similar to that used for fitting a given curve by a series of trigonometric terms in a Fourier series.

Our present purpose requires that this theory be extended to include the case where the supports are no longer stationary but are subjected to an erratic motion such as can be produced by an earthquake. This can be accomplished in the following way: Modify the basic differential equation by adding a term to account for a constant acceleration applied to the supports and solve it subject to the requirement that the beam is undeflected at time zero. This solution is readily obtained by adding a particular integral to the solution previously found for the differential equation applicable to the beam with fixed supports. The particular integral required represents the static deflection of the beam under the steady loads produced by the acceleration. The initial condition is to be met by fitting this curve with a series of the mode functions in the manner previously described. The more general solution required may now be obtained by making use of the law of superposition. To accomplish this, divide the time axis of the earthquake acceleration curve into segments of differential length and sum up the effects which the accelera-

tion increments, originating in each of the above segments, contribute to the motion at a selected time. This process will lead to an integral which is the required solution. It will be found possible to carry out this process even though the moment of inertia and the weight per unit of length are considered to vary with the length, and the stresses are assumed to be proportional both to the strain and to the rate of change of the strain with respect to time. This latter modification will introduce a damping of the "velocity" or "viscous type," and the oscillations will be found to be of a damped sinusoidal type instead of the simple harmonic type previously met. The frequency will also be modified to some extent. Since a frame is composed of beam elements, the methods described will suffice for the determination of earthquake stresses in such a structure also.

NOTATION

The following notation is used:

a_n, coefficient of the "n"th mode.

b, the transverse width of the cross section of a beam at the distance v from the neutral axis.

C_2, C_4, constants.

E, modulus of elasticity.

F, natural frequency of an undamped system.

F_n, frequency of the "n"th mode.

g, acceleration of gravity.

I, moment of inertia of the cross section of a beam.

J, polar moment of inertia of a torsion pendulum disc.

K, viscosity constant.

k_n, a quantity connected with the "n"th mode. See equation 27.

k, the spring constant of an idealized system.

L, length of the vibrating beam.

m, bending moment.

M, weight of the vibrating parts of an idealized system.

R, damping constant of an idealized system.

r, damping constant of the torsion pendulum.

$S(x)$, static load deflection at the point x.

$T(t)$, a function of time only.

t, time.

V, transverse shear in the vibrating beam.

v, a distance measured from the neutral axis in a direction normal to the centerline of the beam.

w, displacement of the vibrating mass of an idealized system with respect to its support.

w_1, displacement of a support of the idealized system with respect to its position at the beginning of the quake.

X, acceleration due to a quake.

x, distance measured along the axis of a beam.

$Y_n(x)$, a deflection pattern for the "n"th mode.

y, displacement of a point on the centerline of a beam with respect to the support.

y_1, displacement of the support of a beam due to an earthquake, measured from the original position of rest.

α_n, a damping constant for the "n"th mode.
β_n, a constant associated with the frequency of the "n"th mode.
γ, a multiplier.
$\gamma_e g$. an equivalent value of transverse acceleration.
ϵ, unit strain.
μ, weight per unit of length of beam.
λ, a time variable running between o and t.
ϕ, angle of rotation of a torsion pendulum disc with reference to the support.
Φ_n, a value of ϕ for the "n"th mode.
ϕ_1, angle of rotation of the support of a torsion pendulum, measured from the original position of rest.
ρ, damping constant for a torsion pendulum.
ψ, spring constant for a torsion pendulum.
ξ, angular acceleration applied to the support of a torsion pendulum.
σ, bending stress in beam.
e, 2.71828+
π, 3.14159+

FORMULAS

Consider a section of the beam as shown in Fig. 1.

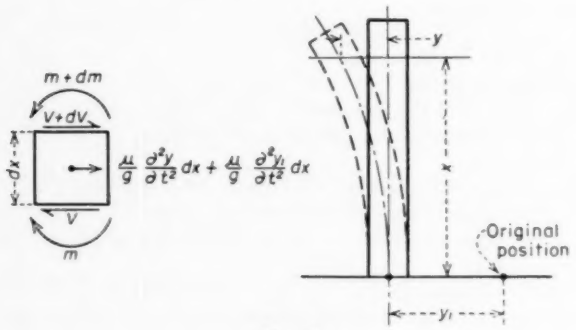

Fig. 1

A point at a distance x from the base is displaced by the amount $(y + y_1)$, where y_1 represents the displacement of the ground with respect to its position prior to the beginning of the quake, and y represents the displacement of a point on the beam with respect to the unstrained position. Newton's law requires that the sum of the forces and the rate of change of momentum be zero, or, if μ represents the weight per unit of length, and g the acceleration of gravity:

$$V - \left(V + \frac{\partial V}{\partial x}\,dx\right) - \frac{\mu}{g}\frac{\partial^2 y}{\partial t^2}dx - \frac{\mu}{g}\frac{\partial^2 y_1}{\partial t^2}\,dx = 0$$

where the symbol t represents time.

This becomes after simplification

$$\frac{\partial V}{\partial x} = -\frac{\mu}{g}\frac{\partial^2 y}{\partial t^2} - \frac{\mu}{g}\frac{\partial^2 y_1}{\partial t^2} \qquad (1)$$

If the rotatory inertia is neglected, equilibrium will require that the sum of the moments shall be zero.

Then,

$$m - \left(m + \frac{\partial m}{\partial x}\, dx\right) + V dx = 0.$$

or

$$V = \frac{\partial m}{\partial x} \qquad (2)$$

Assume the existence of an internal viscosity such that

$$\sigma = E\epsilon + K\frac{\partial \epsilon}{\partial t} \qquad (3)$$

Where σ represents the bending stress, ϵ the unit strain, E Young's modulus, and K the viscosity coefficient.

Then if v represents the distance from the neutral axis of an element bdv of the section

$$\sigma = Ev\frac{\partial^2 y}{\partial x^2} + Kv\frac{\partial^3 y}{\partial x^2 \partial t} \qquad (4)$$

The bending moment will be given by

$$m = \int_{v_1}^{v_2} \sigma b v dv \qquad \begin{matrix} v_2 > 0. \\ v_1 < 0 \end{matrix}$$

where b represents the transverse width of the cross section at the distance v from the neutral axis. Evaluation of the above integral gives

$$m = EI\frac{\partial^2 y}{\partial x^2} + KI\frac{\partial^3 y}{\partial x^2 \partial t} \qquad (5)$$

where I represents the moment of inertia of the cross section. The elimination of V and m from equations (1), (2), and (5) will yield the basic differential equation

$$E\frac{\partial^2}{\partial x^2}\left(I\frac{\partial^2 y}{\partial x^2}\right) + K\frac{\partial^2}{\partial x^2}\left(I\frac{\partial^3 y}{\partial x^2 \partial t}\right) + \frac{\mu}{g}\frac{\partial^2 y}{\partial t^2} = -\frac{\mu}{g}X \qquad (6)$$

where

$$X = \frac{\partial^2 y_1}{\partial t^2}$$

is the ground acceleration.

Solution of the differential equation. The process of solution may be described as follows: A special solution for the case of a frame initially at rest and subjected during the interval $t > 0$ to a uniform transverse acceleration γg is first obtained. This solution is used to construct the general solution by a process to be described later. The special solution will now be obtained.

A particular integral of equation (6) when

$$\frac{\partial^2 y_1}{\partial t^2} = \gamma g$$

is obtainable by integration from the equation

$$E\frac{\partial^2}{\partial x^2}\left(I\frac{\partial^2 y}{\partial x^2}\right) = -\gamma\mu \quad \ldots\ldots (7)$$

The solution of this equation represents the steady deflection of the frame under the influence of the transverse acceleration. Represent the solution of this equation by the expression

$$y_2 = S(x) \quad \ldots\ldots (8)$$

Suppose the transient term to be in the form of a product of functions of x and t only, or

$$y_3 = Y_n(x) \cdot T(t). \quad \ldots\ldots (9)$$

and assume

$$T = -e^{-\alpha_1 t}\left(cos\beta_1 t + \frac{\alpha_1}{\beta_1} sin\beta_1 t\right) \quad \ldots\ldots (10)$$

where α_1 and β_1 are constants to be determined later.

A substitution of these expressions into equation (6) will show that they represent a possible form of solution providing

$$-E\frac{\partial^2}{\partial x^2}\left(I\frac{\partial^2 Y_n}{\partial x^2}\right) + \frac{\mu}{g}(\alpha_1^2 + \beta_1^2) Y_n = 0 \quad \ldots\ldots (11)$$

and

$$-E\frac{\partial^2}{\partial x^2}\left(I\frac{\partial^2 Y_n}{\partial x^2}\right)\frac{\alpha_1}{\beta_1} + K\frac{\partial^2}{\partial x^2}\left(I\frac{\partial^2 Y_n}{\partial x^2}\right)\left(\frac{\alpha_1^2 + \beta_1^2}{\beta_1}\right)$$

$$-\frac{\mu\alpha_1}{g}\left(\frac{\alpha_1^2 + \beta_1^2}{\beta_1}\right) Y_n = 0 \quad \ldots\ldots (12)$$

The solution of equation (11) yields the "mode functions". Since this equation is of the fourth order, its general solution will contain four arbitrary constants. In any case, the attempt to impose appropriate boundary conditions will result in a system of simultaneous equations in which the right-hand members are all zero. Such a system of equations can have no solution other than the trivial one, in which the arbitrary constants are all zero, unless the determinant of the system is itself zero. It is possible to satisfy the requirement in this case since the quantity $(\alpha_1^2 + \beta_1^2)$ is still at our disposal. If this quantity is permitted to vary, a succession of values, called "characteristic values", will generally be found which will reduce the determinant of the system to zero. By using any one of the "characteristic values," it will be found possible to compute the value of any of the unknown arbitrary constants in terms of some selected one of these constants. This means that the mode pattern can be worked out but the scale cannot be fixed. In what follows it will be assumed that the value of the selected unknown constant will be so chosen that the maximum displacement will be unity. The subscript n is used to identify the mode pattern. The successive modes are then designated as the first, second, third, ...nth modes. The additional requirement necessary to satisfy equation (12) will be found to be

$$\alpha_1 = \frac{K}{2E}(\alpha_1^2 + \beta_1^2) \quad \ldots\ldots (13)$$

This equation and the characteristic values can be used to determine the damping rate and frequency for each of the modes.

It remains to satisfy the initial condition that

$$y = 0 \quad \text{when} \quad t = 0.$$

This can be accomplished by choosing a solution of the form

$$y_4 = S(x) + \sum_{n=1}^{n=\infty} a_n Y_n(x)\, T_n(t) \quad \ldots\ldots (14)$$

and determining the quantities a_n to satisfy the requirement

$$S(x) - \sum_{n=1}^{n=\infty} a_n Y_n(x) = 0. \quad \ldots\ldots (15)$$

The values of a_n which will accomplish this can be calculated from the expression (2)

$$a_n = \frac{\int_o^L \mu(x)\, S(x)\, Y_n(x)\, dx}{\int_o^L \mu(x)\, Y^2{}_n(x)\, dx} \quad \ldots\ldots (16)$$

Then the required special solution is represented by equation (14), and it is now possible to use it to construct the more general solution. This is accomplished in the following manner:

Consider an accelerometer record $\frac{d^2y_1}{d\lambda^2} = X(\lambda)$.

During the time interval $d\lambda$ an acceleration increment will come into existence of magnitude $\frac{dX}{d\lambda}\, d\lambda$. Then at the time t the displacement due to this acceleration increment will be

$$dy = \frac{1}{\gamma g} \frac{dX}{d\lambda} d\lambda\, y_4(t-\lambda).$$

The actual displacement at the time t will be the sum of all the displacements produced by the acceleration increments which have come into existence during the interval, or

$$y = \frac{1}{\gamma g} \int_o^t \frac{dX}{d\lambda} y_4(t-\lambda)\, d\lambda \quad \ldots\ldots (17)$$

Since the function X is available only in graphical form, its derivatives are difficult to obtain, and for this reason it is desirable to reduce equation (17) to another form which involves X directly. This can be accomplished by an integration by parts.

If this integration is made subject to the conditions

$y_4 = 0$ when $t = 0$

and

$X = 0$ when $t = 0$

then

$$y = -\frac{1}{\gamma g} \int_o^t X \frac{\partial}{\partial \lambda} y_4(t-\lambda)\, d\lambda \quad \ldots\ldots (18)$$

By substitution, and use of the formula for the sine of the difference of two angles, this expression can be reduced to

$$y = + \sum_{n=1}^{n=\infty} \frac{a_n Y_n}{\gamma g} \left(\frac{\alpha_1^2 + \beta_1^2}{\beta_1} \right) \left[e^{-\alpha_1 t} \sin\beta_1 t \int_o^t X e^{\alpha_1 \lambda} \cos \beta_1 \lambda d\lambda \right.$$

$$\left. - e^{-\alpha_1 t} \cos \beta_1 t \int_o^t X e^{\alpha_1 \lambda} \sin\beta_1 \lambda d\lambda \right] \quad \ldots\ldots (19)$$

This is the solution sought.

Use of the torsion pendulum

The torsion pendulum may be used to evaluate the quantity

$$u = \left(\frac{\alpha_1^2 + \beta_1^2}{\beta_1} \right) \left[e^{-\alpha_1 t} \sin\beta_1 t \int_o^t X e^{\alpha_1 \lambda} \cos\beta_1 \lambda d\lambda \right.$$

$$\left. - e^{-\alpha_1 t} \cos\beta_1 t \int_o^t X e^{\alpha_1 \lambda} \sin\beta_1 \lambda d\lambda \right] \quad \ldots\ldots (20)$$

Since the evaluation of this quantity by approximate integration methods is a very tedious process, use of the torsion pendulum for this purpose will be found advantageous.

The differential equation for the motion of a torsion pendulum suspended from a movable support is:

$$\frac{J}{g} \frac{d^2\phi}{dt^2} + \rho \frac{d\phi}{dt} + \psi\phi = - \frac{J}{g} \frac{d^2\phi_1}{dt^2} \quad \ldots\ldots (21)$$

Let

F represent the undamped natural frequency of the system, and $r = \frac{\rho}{\rho_c}$

where

ρ_c represents the critical damping, or the damping which will just suffice to render the motion of the system non-oscillatory.

Then

$$F = \frac{1}{2\pi} \sqrt{\frac{g\psi}{J}} \qquad \rho_c = 2\sqrt{\frac{\psi J}{g}}$$

and by substitution in equation (21)

$$\frac{d^2\phi}{dt^2} + (4r\pi F) \frac{d\phi}{dt} + (2\pi F)^2 \phi = - \frac{d^2\phi_1}{dt^2} \quad \ldots\ldots (22)$$

A scrutiny of this equation will show that, if the function $\frac{d^2\phi_1}{dt^2}$ is given, the motion depends only on r and F.

Introduce the notation

$$\frac{d^2\phi_1}{dt^2} = \xi \qquad 2\pi F \sqrt{1 - r^2} = \beta_2 \qquad 2\pi r F = \alpha_2$$

then the solution of equation (22) subject to the conditions

$$\phi = 0 \quad \text{when} \quad t = 0$$

$$\frac{d\phi}{dt} = 0 \quad \text{when } t = 0$$

is

$$\phi = -\frac{e^{-\alpha_2 t}}{\beta_2} \sin\beta_2 t \int_0^t e^{\alpha_2 \lambda} \xi \cos\beta_2 \lambda d\lambda + \frac{e^{-\alpha_2 t}}{\beta_2} \cos\beta_2 t \int_0^t e^{\alpha_2 \lambda} \xi \sin\beta_2 \lambda d\lambda \quad \ldots . (23)$$

A comparison of this expression with equation (19) will show that if $\xi = X$, to scale, and in addition $\alpha_1 = \alpha_2$ and $\beta_1 = \beta_2$, then, $\phi = \Phi_n$ and

$$y_n = -\frac{a_n Y_n}{\gamma g} (\alpha_1^2 + \beta_1^2) \Phi_n \quad \ldots . (24)$$

TORSION PENDULUM OPERATION

The torsion pendulum consists of a disc of metal, preferably brass or copper, suspended by a length of piano wire. A disc about ten inches in diameter and one sixteenth inch thick suspended by a two foot length of wire will give good results. The piano wire is attached to a movable support at its upper end in such a manner as will permit the wire to be twisted. An arm attached to the support carries a pointer whose position can be read on a circular scale. An indicating arrow is placed on the edge of the disc and a similar scale is placed below it. These two scales should be identical. A scale of ten divisions per quadrant works well. Damping can be introduced by arranging strong horseshoe magnets in pairs on a transparent glass or celluloid sheet supported just above the metal disc. The arrangement described is directly analogous to the actual case of a structure shaken by an earthquake providing the earthquake displacement is imposed on the support and the period of the pendulum and its damping rate are made to represent those of the structure. In the pendulum arrangement, however, an angular displacement takes the place of the linear one imposed on the structure. Since the earthquake motions would ordinarily be too rapid to follow manually it will be found expedient to alter the time scale in the ratio of about 100 to 1. This is permissible providing the period of the pendulum is increased in the same ratio. The period of the pendulum is readily adjusted by starting with a suspension wire slightly too large and cutting it down with emery cloth until the proper period is obtained. The Coast and Geodetic Survey has computed displacement tables for a number of quakes. With a displacement table at hand the operation of the pendulum proceeds as follows: The pointer is first set to zero on the upper scale, and the pendulum bob brought to rest. The lower scale is then moved to bring the zero opposite the arrow on the disc. An operator and a recorder are required. Time correlation is provided by means of two stop watches which are started simultaneously at the beginning of the run. The operator adjusts the

support at frequent intervals, usually every five seconds, and the recorder observes the position of the pendulum simultaneously. However, in order to give the recorder time to write down his observations he skips alternate readings and records every ten seconds instead. A typical set of such readings is shown in Table 1. The position of the pendulum with respect to the support is obtained by subtracting the pendulum displacement from the quake displacement. To obtain good results it is necessary to adjust the support at sufficiently frequent intervals to describe the quake motion closely. It is also necessary to make the intervals small compared to the period of the pendulum. A program of at least ten adjustments per period should be adhered to. A run is made for each mode frequency under consideration.

TABLE 1

Frequency 1.00 cycle per second — No damping
E-W Trace of the Helena, Montana, Quake of Oct. 31, 1935
Time interval—5 seconds — Read every 10 seconds
Total time—8 minutes 20 seconds — T = 87.0 seconds

t	P	Q	Q-P	t	P	Q	Q-P
0	0	0	0				
10S	4	33	29	4M-40S	502	364	−138
20	15	67	52	4 -50	218	369	151
30	48	101	53	5 -00	20	378	358
40	102	136	34	5 -10	−2	385	387
50	168	166	−2	5 -20	168	394	226
1M-0S	227	199	−28	5 -30	462	405	−57
1 -10	265	224	−41	5 -40	705	401	−304
1 -20	282	239	−43	5 -50	815	396	−419
1 -30	275	253	−22	6 -00	715	388	−427
1 -40	254	266	12	6 -10	462	379	−83
1 -50	238	280	42	6 -20	150	375	225
2 -00	242	299	57	6 -30	−25	370	395
2 -10	272	325	53	6 -40	−8	367	375
2 -20	324	364	40	6 -50	203	368	165
2 -30	398	372	−26	7 -00	490	366	−124
2 -40	455	299	−156	7 -10	718	354	−364
2 -50	450	223	−227	7 -20	763	346	−417
3 -00	352	150	−202	7 -30	598	341	−257
3 -10	168	109	−59	7 -40	297	341	44
3 -20	−30	82	112	7 -50	32	341	309
3 -30	−182	120	302	8 -00	−83	335	418
3 -40	−180	208	388	8 -10	12	329	317
3 -50	5	237	232	8 -20	260	325	65
4 -00	295	303	8				
4 -10	620	348	−272	Scale constant = −0.000,498,68			
4 -20	738	352	−386	Max. defl. = −427 (−0.000,498,68) =			
4 -30	702	346	−356	+0.213 ft.			

P pendulum displacement
Q quake displacement
t time
T pendulum period

Treatment of damping

The equations in the preceding paragraphs are derived on the assumption that the damping is of the internal viscous type, even though it is probably true that of all the types of damping which may be expected to occur, this type is least apt to be present. In actual practice the natural damping will generally be due to some combination of the following causes.

(1) Elastic hysteresis.

(2) Mechanical friction.

(3) External viscous damping due to contact with water or air.

(4) External square law damping due to eddy formation in water or air.

At first glance the above statements would appear to constitute a serious criticism of the validity of the equations presented, but such is not necessarily the case. It is to be expected that a knowledge of damping characteristics will come from tests made upon actual structures of a type similar to those under consideration, and that the data will be in such form as will not permit identification of the cause. Under these conditions engineering purposes will be best served by introducing the simplest equations which will permit evaluation of the damping effect, and this is the course which has been followed herein. If the values of α are chosen to represent the actual damping rates, the difficulties described should not prevent a satisfactory evaluation of the effect of damping.

Sample computation

The methods described above will now be illustrated by application to a cantilever beam. If the origin is at the fixed end, the static load curve which will satisfy equation (7) is given by the expression:

$$S = -\frac{\gamma\mu}{24EI}\left[x^4 - 4x^3L + 6x^2L^2\right] \quad \ldots\ldots (25)$$

where γg represents the transverse acceleration. The mode functions which satisfy the differential equation, and the conditions that the slope and deflection are zero at the fixed end, and that the moment and shear are zero at the free end are of the form:[1]

$$Y_n = C_2\,(\cos k_n x - \cosh k_n x) + C_4\,(\sin k_n x - \sinh k_n x) \quad \ldots\ldots (26)$$

where

$$k_n = \sqrt[4]{\frac{\mu(\alpha_n^2 + \beta_n^2)}{gEI}} \quad \ldots\ldots (27)$$

and

$$C_2 = -\left(\frac{\cos k_n L + \cosh k_n L}{2 \sin k_n L \sinh k_n L}\right) Y_1 \quad \ldots\ldots (28)$$

$$C_4 = -\left(\frac{\sin k_n L - \sinh k_n L}{2 \sin k_n L \sinh k_n L}\right) Y_1 \quad \ldots\ldots (29)$$

where $Y = Y_1$ when $x = L$.

The first three characteristic values are: (1) (2)

$$k_1L = 1.87510$$
$$k_2L = 4.69410$$
$$k_3L = 7.85476$$

The mode amplitudes can now be computed from equations (16), (25), and (26). They are:

$$a_1 = \frac{2\gamma\mu L^4}{EIY_1}(-0.06334) \quad \ldots\ldots (30)$$

$$a_2 = \frac{2\gamma\mu L^4}{EIY_1}(+0.000894) \quad \ldots\ldots (31)$$

$$a_3 = \frac{2\gamma\mu L^4}{EIY_1}(-0.000066) \quad \ldots\ldots (32)$$

It is worthwhile to note here that since Y_1 occurs in the denominator of the above quantities it exerts no influence on the products $a_n Y_n$. This means that the value of Y_1 can be chosen arbitrarily to suit the convenience of the computer.

The frequencies can be computed with the aid of equation (27) which can be put in the form

$$\alpha_n^2 + \beta_n^2 = \frac{gEI}{\mu L^4}(k_nL)^4 \quad \ldots\ldots (33)$$

If an appropriate value of K is known the value of α_n can be computed by means of equation (13), but since damping may be due to a combination of causes it may be preferable to determine α_n by use of data obtained from structures similar to the one under consideration. When the value of α_n has been fixed, the value of β_n is readily found. The natural frequency is then obtained from the relation:

$$F_n = \frac{\beta_n}{2\pi} \quad \ldots\ldots (34)$$

The numerical values can now be used. These will be chosen to represent appropriate values for the Boulder intake towers.

Let $I = 985{,}000$ ft.^{4} (approximate value at the base)
$L = 340$ ft.
$\mu = 708{,}000$ lb/ft. (approximate average value)*
$E = 576{,}000{,}000$ lb/ft^2.
$\gamma = 1.0$

choose

$Y_1 = 1.0$ ft.

then

$$\frac{2\gamma\mu L^4}{EIY_1} = 33.35$$

$$a_1 = -2.11$$
$$a_2 = +0.0298$$
$$a_3 = -0.0022$$

for the present purposes assume $\alpha_1 = \alpha_2 = \alpha_3 = 0$ which is the condition for no damping. The frequencies can now be computed. These are:

*This quantity is made up of the following items:

Concrete	272,000 lb. per ft.
Water between fins and inside the shaft	162,000 lb. per ft.
Water outside the fins which may be considered to move with the tower	274,000 lb. per ft.
Total	708,000 lb. per ft.

For means of computing the last item see par. 68 of the sixth edition of Lamb's "Hydrodynamics."

$F_1 = 0.7776$ cycles/sec.
$F_2 = 4.873$ cycles/sec.
$F_3 = 13.64$ cycles/sec.

The bending moment at the base of the tower will be computed for purposes of illustration, but the same methods would apply elsewhere in the structure. The bending moments can be obtained by differentiating equation (24) twice with respect to x and multiplying by EI. Then, if m_n represents the bending moment for the nth mode,

$$m_n = -\frac{a_n EI Y''_n}{\gamma g}(\alpha_1^2 + \beta_1^2)\,\Phi_n \quad \ldots\ldots\ldots\

Fig. 3—Improved earthquake analyzer constructed at the Bureau of Reclamation

The earthquake displacement record runs over a drum under the rectangular window below the diagonal mirror at the top of the machine. An operator follows the record with a pointer by turning one of the pairs of knobs. A second operator follows the motion of the pendulum by turning the second pair of knobs. To permit this an image of the pendulum is brought up through the round window under the second diagonal mirror. The required subtraction is automatically made and the response curve drawn on a chart. Damping is applied by magnets acting on the pendulum disc.

Let

w represent the displacement of the weight with reference to the support

and

w_1 the displacement of the support due to the quake measured from the initial position of rest.

Then the differential equation of motion will be

$$\frac{M}{g}\frac{d^2w}{dt^2} + R\frac{dw}{dt} + kw = -\frac{M}{g}\frac{d^2w_1}{dt^2} \quad \ldots\ldots (37)$$

where R represents the damping constant.

The substitutions

$$F = \frac{1}{2\pi}\sqrt{\frac{gk}{M}} \qquad R_c = 2\sqrt{\frac{kM}{g}} \quad \text{and} \quad r_1 = \frac{R}{R_c}$$

will immediately reduce this equation to the form

$$\frac{d^2w}{dt^2} + (4r_1\pi F)\frac{dw}{dt} + (2\pi F)^2 w = -\frac{d^2w_1}{dt^2} \quad \ldots\ldots (38)$$

A comparison of these equations with equations (21) and (22) will show that they are analogous expressions for linear and rotary motions.

When the oscillation reaches its maximum amplitude the velocity becomes zero and the above expression then assumes the form

$$(2\pi F)^2 w_{max} = -\frac{d^2}{dt^2}(w + w_1) \quad \ldots\ldots (39)$$

The right-hand member of this expression represents an acceleration, and since the stress in the elastic members which supply the restoring force is proportional to the

TABLE 2

E-W Trace of Helena, Montana, Quake of Oct. 31, 1935.

$$\gamma_e = \frac{(2\pi F)^2 \, \Phi_{max}}{32.2} \times 100. \quad \text{(in \%)}$$

F	$(2\pi F)^2$	Φ_{max}	γ_e *in %*
0.50	9.87	−0.2132	6.53
0.75	22.2	0.2007	13.8
1.00	39.5	0.2092	25.6
1.25	61.7	−0.1052	20.1
1.50	88.8	−0.0770	21.2
1.75	120.9	−0.0473	17.8
2.00	157.9	−0.0558	27.4
2.25	199.9	0.0532	33.0
2.50	246.7	−0.0678	52.0
2.75	298.6	0.0558	51.7
3.00	355.3	0.0531	58.6
3.25	417.0	0.0411	53.2
3.50	483.6	0.0461	69.2
3.75	555.2	0.0428	73.8
4.00	631.7	−0.0351	68.9
4.25	713.1	−0.0225	49.8
4.50	799.4	0.0311	77.2
4.75	890.7	0.0241	66.7
5.00	987.0	−0.0257	78.8

displacement w, the left-hand member can be seen to represent the maximum equivalent acceleration applied to the structure during the oscillation. Then the maximum equivalent acceleration applied to the system during the quake can be obtained by multiplying the greatest displacement by the quantity $(2\pi F)^2$.

This result can be expressed as a percentage of gravity by dividing by the acceleration of gravity and multiplying by 100. Table 2 contains a series of such values computed for the east-west component of the Helena, Montana quake of October 31, 1935.

The working of this approximate method will be illustrated by applying it to the intake tower data.

The weight of the tower is

$$M = (708{,}000)\ (340) = 240{,}720{,}000 \text{ lb.}$$

Since the center of mass of a uniform cantilever is at midheight, the spring constant will be estimated from the formula for the deflection of a cantilever with a concentrated load at the end. In this case the cantilever will have a length equal to half the tower height.
Then

$$k = \frac{3EI}{(L/2)^3} = (346.4)\ (10)^6$$

pounds to produce a deflection of one foot,
and

$$F = \frac{1}{2\pi}\sqrt{\frac{gk}{M}} = \frac{1}{2\pi}\sqrt{\frac{(32.2)\ (346.4)\ (10)^6}{(240.72)\ (10)^6}}$$

or

F = 1.08 cycles per second.*

This value corresponds approximately to the first mode frequency of 0.7776 cycles per second previously obtained. The nearest tabular value is for 1.0 cycles per second, and the equivalent transverse acceleration from Table 2 would be 25.6 per cent gravity. The maximum bending moment at the base would then be

$$\frac{(240.72)\ (10)^6 \quad (340)\ (0.256)}{2} = (10.48)\ (10)^9 \text{ ft.lb.}$$

This value would compare with the (7.5) (10)^9 foot-pounds of bending moment previously obtained. Since the maximum recorded acceleration during the quake was about 15 per cent gravity for this component, the value of 25.6 per cent gravity obtained represents an evaluation of the effect of resonance.

A comparison of the distribution of bending moments obtained by this approximate procedure with those previously obtained would show that, while it gives too large moments at the base, the computed moments at the higher elevations would be less than they should be. The reason for this is to be found in the response in the second mode which will produce strong bending moments at about midheight.

The limitations of the method of providing for earthquake forces by designing the structure to withstand a transverse load equal to a fixed percentage of the weight can now be seen. The limitations of this method are essentially those of the approximate procedure just described. This will work well if the structure actually approximates the idealized case of a concentrated mass held in position by a weightless spring, but if the restoring force and the weight are both distributed, as they are in a frame, then there may be vibrations in several modes, and the bending moment distribution obtained by the approximate method may be unreliable.

CONCLUSIONS

The conclusions arrived at as a result of these studies may be summarized as follows:

(1) A solution has been found for the case of a framed structure subjected to an earthquake.

(2) The effect of damping can be accounted for.

(3) Data obtained from experiments performed on a torsion pendulum can be substituted for data obtained from computation with considerable saving in time.

(4) Either an accelerometer record or a seismograph record can be used as a basis for the computation.

(5) The methods developed make it possible to plot the time history of the response of all points in a frame structure, subjected to any quake for which an accelerometer or seismograph record is available.

(6) A frame will vibrate in a number of distinct patterns or "modes" each of which has its own associated natural frequency.

(7) The use of an earthquake allowance in the form of a transverse load which is a specified constant fraction of the weight of the structure will

*The Rayleigh method can also be used for estimating the frequency. For a description of this method, see paragraph 14 of Timoshenko's "Vibration Problems in Engineering," second printing, p. 55.

give reliable results only if the structure actually approximates to the idealized system consisting of a concentrated mass held in position by a weightless spring.

(8) If the mass and restoring forces are both distributed, as is the case in a frame, a response in several modes may be obtained, and the resulting moment distribution will not correspond to that obtained for any transverse load which is a fixed proportion of the weight.

(9) If the structure approximates the idealized system having one degree of freedom, the use of a transverse load which is a fixed proportion of the weight is proper, and the factor of proportionality for any quake can be obtained by use of the torsion pendulum.

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REFERENCES

1. "Vibration problems in Engineering" by S. Timoshenko.
2. "Mechanical Vibrations" by Den Hartog.
3. "Theory of Sound" by Rayliegh.
4. "Hydrodynamics" by Lamb.
5. "An Improved Method for Calculating Free Vibrations in Systems of Several Degrees of Freedom" by Winston M. Dudley in The Journal Of Applied Mechanics for June 1938.
6. "Matrices in Engineering" by Louis A. Pipes in Electrical Engineering for September 1937.
7. "Applied Elasticity" by J. Prescott.
8. "Earthquake Studies for Pit River Bridge" by J. L. Savage in Civil Engineering for August 1939.
9. "United States Earthquakes 1935". Coast and Geodetic Survey publication No. 600.
10. "Strong Motion Report No. 4." Coast and Geodetic Survey.

NOTE: Mr. Glover calls attention to the fact that while his present paper was being prepared for publication two other papers have appeared, both by M. A. Biot:

"A Mechanical Analyzer for the Prediction of Earthquake Stresses," Bulletin, Seismological Society of America, Vol. 31, No. 2, Apr. 1941, p. 151 to 171, and
"Analytical and Experimental Methods in Engineering Seismology", *Proc.* ASCE, Jan. 1942, p. 49.

Discussion of this paper should reach the ACI Secretary in triplicate by July 15, 1942, for publication in the JOURNAL Supplement, concluding Proceedings V. 38.

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Discussion of a paper by Robert E. Glover:

Earthquake Stresses in Frame Structures*

DISCUSSION BY MERIT P. WHITE, ERIK SOLLID AND THE AUTHOR

By MERIT P. WHITE†

The problem of designing earthquake resistant structures is a very important one in many parts of the world. It is also a very difficult and complicated problem. Present design methods are empirical, and are justified mainly by the fact that they are known from experience to give earthquake resistance. Efforts, such as that of Mr. Glover, to supply a sound analytical, rather than empirical basis for earthquake design, are worthwhile and should be continued.

The extreme complexity of the problem of seismic design has two sources. One of these lies in the complicated nature of ground motions, and in the fact that no two earthquakes are alike. The second is the fact that even the simplest structure is a tremendously complicated mechanical system. The existence of distributed internal friction and yielding supports makes the situation worse.

Fortunately, these two complicating factors can be treated separately. Any structure, even the most complex, has certain normal modes of vibration. Each of these has a definite shape, frequency, and rate of energy dissipation. The response of a structure to a disturbance will be the sum of the responses of its various modes. It can be shown that, to a close approximation, there is a definite relation between the response of a particular mode of a particular structure and the response of a simple, one degree of freedom oscillator with the same period and the same degree of damping. The response of a simple oscillator to a particular disturbance can be found either experimentally, as suggested by the author, or analytically[1] without much difficulty.

The experimental method which the author describes consists in giving to the support of a torsional pendulum an angular displacement propor-

*ACI JOURNAL, Apr. 1942; *Proceedings* V. 38, p. 453.
†Dept. of Civil Engineering, Illinois Institute of Technology, Chicago.
[1]See references at end of discussion.

tional to the linear displacement of the shock being studied. The response of the pendulum may be measured. The principal objection to this method lies in the fact that all usable strong motion records are acceleration records which must be integrated twice to give displacements. This double integration is a difficult and tedious task[2]. In his *Conclusions* the author states that an accelerometer record can be used with the computation method. This is true, of course. It is also true that an accelerometer record can be used to govern the displacement of the base of the torsion pendulum used in the experimental method. The resulting pendulum response can be easily used to find the response which would result if the ground displacement were used in place of the acceleration. The substitution of acceleration for displacement records in model studies is described in a paper by Ralph Byrne and the writer[3].

The title of the present paper is somewhat misleading since the analysis contained therein applies directly only to the case of a cantilever beam which is flexible in bending and is supported on an unyielding base. Comparatively few frame structures can be so represented. It is implied (see end of *Introduction*) that since a frame is composed of beam elements the present analysis can be applied to frame structures. The difficulty of such an undertaking can only be described as colossal. Furthermore, the presence of floors, walls, and interior partitions will so alter the characteristics of a frame that any results obtained in this manner will be without significance.

Internal friction undoubtedly has a very important effect on the response of a structure to an earthquake[4]. The author lists possible sources of friction in a building and proposes the use of a viscous or linear friction law to care for the damping effect. The little evidence available indicates that a linear damping is the best approximation to the actual friction mechanism[3, 5]. Of course there are numerous types of linear damping. The author introduces viscous stresses proportional to rate of change of strain (Equation 3). Another possibility is to consider external forces proportional to rate of change of displacement. The available evidence indicates that for a typical building both assumptions are about equally in error, and that the true situation is between them[1].

On the basis of the facts mentioned, the writer suggests that no portion of the earthquake problem can be treated by pure theory, unless enough simplifying assumptions are made to completely invalidate the conclusions. The writer believes, however, that it is possible so to combine theory with experiment as to give results of engineering value.

If the shapes, frequencies, and damping coefficients of the lower modes of vibration of a structure are known, together with the distribution of weight, then the inertia forces in this structure caused by any disturbance

can be calculated. For certain idealized types of structures the mode shapes can be found analytically without great effort. Mode shapes have been determined experimentally for many existing structures by use of the building vibrator constructed by Stanford University and the USCGS[5]. With experience, it is possible in many cases to assume mode shapes with enough accuracy for engineering calculations.

Natural frequencies have also been obtained by means of the building vibrator. A comparison of these with the frequencies calculated from knowledge of the frame stiffness and weight distribution shows great disagreement, due to the fact that walls and partitions furnish the greatest part of the stiffness of an ordinary building. However, it is possible to predict the lower frequencies of a proposed structure from knowledge of the frequencies of similar structures on similar sites.

The damping in existing structures can also be measured by use of the vibrator, although few such investigations have been made[3]. When more information on this subject is available it will be possible to predict accurately enough the damping factors for a proposed building.

Consider a typical, approximately symmetrical building. Using the author's notation, let the weight distribution be represented by the function $\mu(x)$, where x is distance measured from the base. The lower mode shapes are found analytically or experimentally and are represented by the functions $Y_n(x)$, where $n = 1, 2, 3 \ldots$, indicating mode order. As in the author's analysis, the scale of the Y_n's is unimportant, only the *mode shapes* are significant. Since the Y_n's represent independent modes of vibration they constitute, to a first approximation, a complete set of functions which are orthogonal with respect to the weight function $\mu(x)$. That is

$$\int_0^L \mu Y_n Y_m dx = 0 \qquad \text{if } n \neq m \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

In addition, the frequencies $f_n = \dfrac{\beta_n}{2\pi}$ and the damping coefficients α_n associated with the different vibration modes are assumed to be known.

Let this system be subjected to a base motion consisting of a constant velocity Δu which begins suddenly at time $t = 0$. The resulting deflection (not displacement) of any point of the structure at any time can be represented by the expressions:

$$\left.\begin{aligned} y(x, t) &= 0; \qquad t \gtrless 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \\ y(x, t) &= \Sigma\, Y_n(x) e^{-\alpha_n t} (a_n \sin \beta_n t + b_n \cos \beta_n t); \; t \lesseqgtr 0 \ldots\ldots\ldots \\ n &= 1, 2, 3 \ldots \end{aligned}\right\} \quad (2)$$

The coefficients a_n and b_n can be found from the requirement that at time $t = 0$,

$$y(x, t) = 0$$

$$\frac{\partial y}{\partial t} = -\Delta u$$

It can be shown that all the b_n-coefficients must equal zero, while the a_n's are determined by the relation

$$-\Delta u = \Sigma\, \beta_n a_n Y_n(x).$$
$$n = 1, 2, 3 \ldots .$$

Multiplying both sides of the last equation by $\mu(x) Y_n(x) dx$ and integrating over the height of building yields

$$a_n = -\frac{\Delta u}{\beta_n} \frac{\int_0^L \mu Y_n dx}{\int_0^L \mu Y_n^2 dx} \qquad (3)$$

For a velocity increment beginning at time $t = \lambda$ instead of $t = 0$, the response is found by replacing t in Equation 2 by $t - \lambda$. Then, combining expressions 2 and 3, there is obtained:

$$y(x, t) = 0; \quad t \lesssim \lambda$$

$$y(x, t) = -\Sigma\, Y_n(x) \frac{\int_0^L \mu Y_n dx}{\int_0^L \mu Y_n^2 dx} \frac{\Delta u}{\beta_n} e^{-\alpha_n(t-\lambda)} \sin \beta_n(t-\lambda); \quad t \gtrsim \lambda$$
$$n = 1, 2, 3 \ldots$$

The last expression can be generalized to care for any kind of base motion simply by treating the latter as a series of velocity increments Δu which occur at successive instants λ. At any time t, the total response of the system will be the cumulation of the effects of all the impulses occurring up to that time. The disturbance is assumed to begin at $t = 0$. Since $\Delta u = \frac{du}{d\lambda}\Delta\lambda = X\Delta\lambda$, where X = earthquake acceleration, the general expression becomes

$$\left.\begin{aligned} y(x, t) &= \Sigma\, Y_n(x) \frac{\int_0^L \mu Y_n dx}{\int_0^L \mu Y_n^2 dx} \left\{ -\frac{1}{\beta_n} \int_0^t X e^{-\alpha_n(t-\lambda)} \sin \beta_n(t-\lambda) dx \right\} \\ & \qquad n = 1, 2, 3 \ldots . \\ &= \Sigma\, Y_n(x) \frac{\int_0^L \mu Y_n dx}{\int_0^L \mu Y_n^2 dx} R_n(t) \ldots \ldots \ldots \ldots \\ & \qquad n = 1, 2, 3 \ldots \end{aligned}\right\} \qquad (4)$$

The quantity $R_n(t)$ is the deflection of a simple oscillator whose frequency

and damping correspond to the nth mode of the structure, caused by the disturbance X.

Ordinarily, the transverse shear force will be the critical load on a building. The shear at the base of the structure is obtained by summing up the inertia forces over the whole system. It can be shown that the shear force at the base is given approximately by the expression:

$$\left.\begin{aligned} H(t) &= \sum \frac{\left[\int_0^L \mu Y_n dx\right]^2}{g \int_0^L \mu Y_n^2 dx} \left(R_n \beta_n^2\right) \\ &\qquad n = 1, 2, 3, \ldots \\ &= \Sigma\, A_n (R_n \beta_n^2) \\ &\qquad n = 1, 2, 3, \ldots \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

For any structure whose modes have been determined, and for any earthquake shock, the above expression can be evaluated. Note that each term in the series is the product of two factors, one, A_n, depending only on the characteristics of the structure, the other, $R_n\beta_n^2$, representing approximately the shear in a simple oscillator of known frequency and damping and of unit mass, caused by the given ground motion. It has been found that the first term of the above series, representing the contribution of the fundamental mode, is ordinarily the most important.

It is interesting to compare values of the A-coefficients for different types of structures[1]:

1. Concentrated mass (the simple oscillator)
   $A_1 = 1$; $A_2 = A_3 = \ldots = 0$.

2. Structure of uniform mass and shear stiffness on a rigid base. The modes are represented by the functions $Y_n = \sin \dfrac{(2n - 1)\pi x}{2L}$. Then it can be shown that
   $A_1 = 0.81$; $A_2 = 0.09$; $A_3 = 0.03$ . . .

3. Mass and shear stiffness both vary linearly from top to base. Foundation is rigid. The mode shapes are given by Bessel Functions and
   $A_1 = 0.69$; $A_2 = 0.13$; $A_3 = 0.05$ . . .

4. Mass is uniformly distributed. Fundamental mode deflection is proportional to x. This may correspond to a very rigid structure on a yielding base, or to a structure in which shear and bending flexibilities are about equally important. Then
   $A_1 = 0.75$

5. The same as above except that fundamental mode deflection is parabolic, proportional to x^2. Then
   $A_1 = 0.55$

It has been shown that the response of any structure to an earthquake can be expressed in terms of the responses of simple damped oscillators to the motion in question. This fact will greatly simplify the investigation of the general earthquake problem. As strong motions occur and are recorded, the responses of simple oscillators of different periods, and degrees of damping can be found. When enough shocks have been analyzed in this manner it will be possible to construct curves of expected intensities in terms of oscilator period and damping, and location. Finally, any proposed structure whose fundamental period, damping and mode shape are known approximately, can be designed for definite lateral forces. In this way it will be possible to obtain more consistent design than is possible by present methods.

REFERENCES

(1) See "Earthquakes and their Effects on Structures," a report by R. R. Martel and Merit P. White to the Department of Building and Safety of Los Angeles County, Calif., 1939, unpublished.

(2) See Chapter 3, "The Analysis of Records" by Frank Neumann, of "Earthquake Investigations in California, 1934-1935," Special Publication 201 of U. S. Dept. of Commerce.

(3) "Model Studies of the Vibrations of Structures during Earthquakes" by Merit P. White and Ralph Byrne, Bulletin of the Seismological Society of America, V. 29, Apr. 1939.

(4) "Friction in Buildings," Merit P. White, Bulletin of the Seismological Society of America, V. 31, Apr. 1941.

(5) See Chapter 7, "The Building and Ground Vibrator" by J. A. Blume and L. S. Jacobsen, of "Earthquake Investigations in California, 1934-1935," Special Publication 201 of U. S. Dept. of Commerce.

By ERIK SOLLID*

The writer wishes to emphasize the importance of the relation between the natural vibration period of a structure to the nature of the earthquake motion. The objection to using a horizontal design force as a fixed portion of the weight could be overcome easily by using different proportions of the weight at different elevations. Also, he cannot agree with the author that damping can be accounted for entirely merely by introducing a fixed "damping rate." In cases of slipping of joints, yielding of foundation, yielding of locally overstressed parts, a great amount of energy is absorbed without adding anything to the kinetic energy of vibration; hence, the period of vibration of the structure is changed. The relation between the natural vibration period of the structure and the vibration of the ground during an earthquake is, no doubt, the largest single factor determining stress in a structure subject to an earthquake. Therefore, damping, such as mentioned above, has perhaps its biggest effect by

*Bureau of Reclamation, Denver, Colo.

preventing synchronism between the vibration period of the structure and that of the ground.

C. S. Newell and the writer[1] approached the problem of stresses in structures caused by earthquakes by calculating deflections resulting from inertia forces acting during forced vibration. From the deflection at any time, expressions for inertia forces were evolved. This work was limited to the fundamental mode of vibration, and the effect of damping was also here introduced merely as a "damping rate." (For detail discussions and derivation of equations, the reader is referred to reference 1.)

From Fig. A to C, the following relation between the structure and the ground acceleration is easily set up.

$$\frac{d^2s}{dt^2}+\frac{u}{m}\frac{ds}{dt}+\frac{k}{m}s = \frac{d^2x}{dt^2} = \text{acceleration of the ground motion} \quad \ldots (1)$$

where

s = deflection of mass center of vibration of the structure,
u = frictional force, in pounds per unit velocity of deflection,
m = mass,
k = spring constant of a structure expressed in pounds per foot deflection, s,
t = time.

For purpose of illustration, the right-hand member of equation 1 was first set equal to the acceleration of an idealized earthquake motion following the laws of simple harmonic motion; that is,

$$\frac{d^2x}{dt^2} = zw^2 \sin wt \quad \ldots (2)$$

where

z = maximum displacement of the ground during an assumed earthquake of simple harmonic motion,
w = angular velocity of simple harmonic earthquake motion.

Letting u_c be the value of u for critical damping (critical damping is such that when a structure is deflected from its position of rest and released, the structure will move back to this position without vibrating back and forth) and letting the period of earthquake motion be the same as the natural period of vibration of the structure, the maximum deflection the structure would suffer may be expressed by

$$s_{max} = z\frac{w^2m}{k}\frac{u_c}{2u} \quad \ldots (3)$$

[1]Technical Memorandum No. 583, "Method of Calculating Deflections of a Structure Subject to a Known Earthquake," Nov. 8, 1938, Bur. of Rec., Denver, Colo.

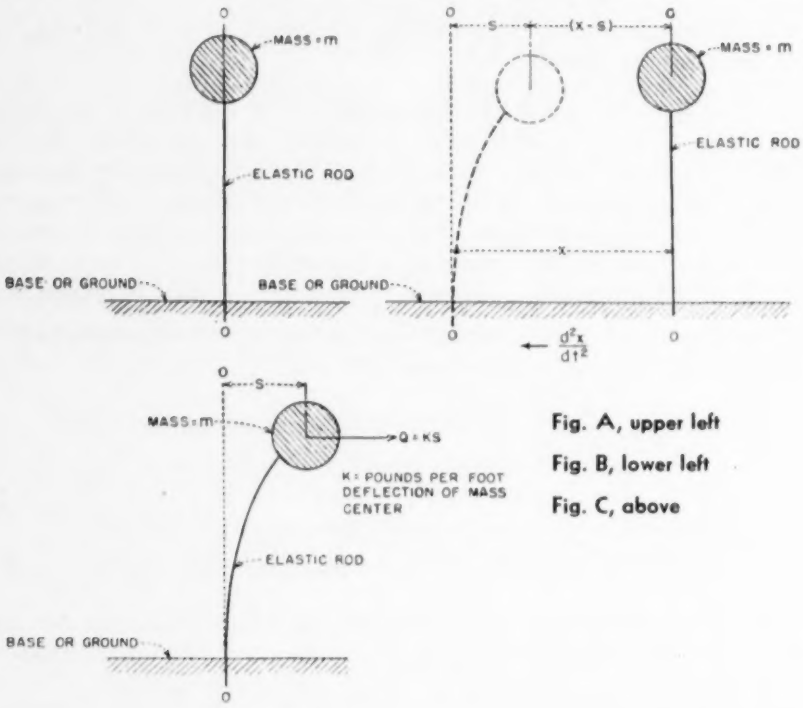

Fig. A, upper left

Fig. B, lower left

Fig. C, above

Equation 3 shows that even at reasonance the deflection will not build up indefinitely but will approach a definite value, depending on the degree of damping. It is agreed that here the damping could be represented by a fixed "damping rate." However, as soon as the damping is caused by slipping of joints, or by any inelastic yielding, the synchronism is broken and the relation of equation 3 is not valid. It is therefore evident that the deflection, hence the stress, would be different than it would be were it based on a fixed damping as used in equation 3. This is also true for the higher modes of vibration.

Inspection of the records of past major earthquakes shows that the accelerograms consist virtually of straight lines between certain time limits. It is therefore convenient to divide the accelerograms into parts, one for each straight portion, as shown dotted and marked in Fig. D. Let a denote acceleration at the beginning of any straight portion, and let c denote the slope. Then the acceleration of the earth at any time t will be

$$\frac{d^2x}{dt^2} = a + ct \quad \ldots\ldots\ldots\

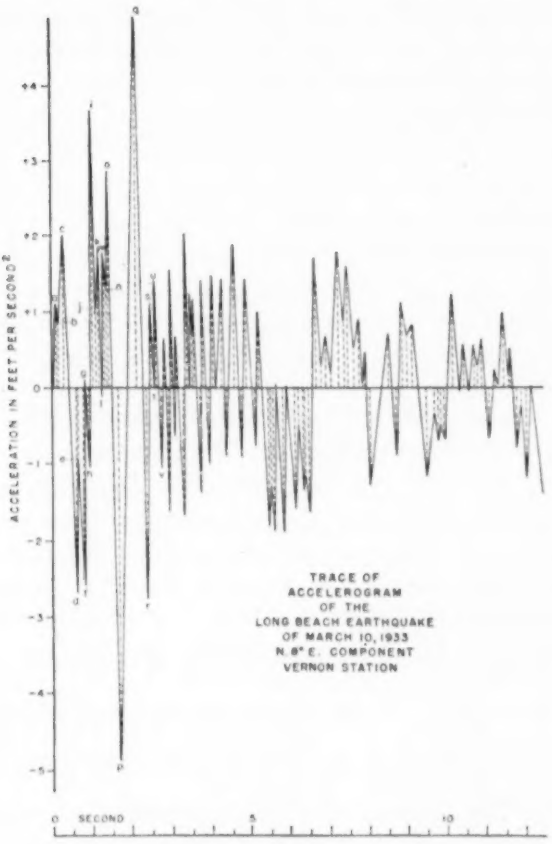

Fig. D

where t is measured from the beginning of each straight portion. Inserting in equation 1 and solving, gives

$$s = e^{-\frac{u}{2m}t}\left(A \sin pt + B \cos p\,t\right) + \frac{1}{p^2}(a + c\,t) - \frac{u}{m}\frac{c}{p^4} \quad \ldots\ldots (5)$$

where

$e = 2.718{,}282$ - - -

p = angular velocity of simple harmonic motion of a structure,

A and B = arbitrary constants.

By means of equation 5, for any given structure and earthquake, the deflection s of the structure at any time may be computed. However, it is subject to the restriction that the damping is introduced as a "damping rate."

From the maximum deflection, the maximum inertia force may be evaluated. Expressing this force as a part α of the weight or gravitation, the value of α may be expressed by

$$\alpha = s_{max.} \frac{p^2}{g} \quad \ldots\ldots (6)$$

where

g = acceleration of gravity,
$s_{max.}$ = maximum value of deflection at any point.

This equation is not exact, but it is close enough for the present purpose of illustrating that α is a variable, since $s_{max.}$ varies for various points. The use of a horizontal design force computed as $\alpha\, g\, m$ is the simplest method available to the designer. It is therefore suggested that in a practical way any load pattern could be approximated by using different values of α between different elevations in a structure.

Messrs. J. N. LeConte and J. E. Younger[2] have also derived equations for moments and shears in a structure caused by earthquakes, taking into account the higher modes of vibration. However, no attempt was made to evaluate the effect of damping and only an idealized earthquake of simple harmonic motion was used. These equations for moments and shears contain the ratio $\frac{T}{T_0}$ where T is the natural period of vibration of the structure and T_0 is the period of simple harmonic earthquake motion. This is analogous to the term $\frac{1}{p^2}\ (a + c\,t)$ of equation 5. Since the period T_0 or the quantity $(a + c\,t)$ is unpredictable, the problem of determining inertia loads during an earthquake resolves itself into a probability problem. At best, one can but work out probability curves based on records of past earthquakes. The use of a torsion pendulum as constructed by the author or by M. A. Biot would be very helpful in evaluating such curves.

AUTHOR'S CLOSURE

In his discussion, Mr. Sollid objects to the use of a fixed damping rate because the effect of slipping joints, foundation yielding, and yielding due to overstress can not be adequately accounted for by using it. The author would prefer to restrict the term damping to those factors which cause a wasting of energy of vibration without changing the nature of the structure, and to make such important factors as those mentioned above the subject of a special investigation. A promising method of attack for such cases has been demonstrated by W. A. Waldorf, who is

(2) Bulletin of the Seismological Society of America, V. 22, No. 1, March 1932.

now absent on a furlough from the Bureau of Reclamation. He constructed a torsion pendulum with a supplementary device which permits an inelastic yielding beyond the elastic limit to be accounted for. An evaluation of the effects of slipping joints or foundation yielding could probably also be made by this procedure.

Mr. Sollid suggests the use of variable mass factors to improve the form of the load distribution curves. This device is of value in those types of vibration in which all parts of the structure move in the same direction. In fact, the fundamental mode pattern is generally obtained if the masses are weighted in proportion to the displacements and the sign of these is kept everywhere the same. Load patterns with reversals of sign, such as normally occur in the higher modes, would seem however, to be beyond reach of the method, unless negative proportionality factors were permitted. If such factors were permitted the suggested procedure would resolve itself into a computation of mode patterns.

Mr. White points out that there is an alternative torsion pendulum operation procedure available which makes use of an acceleration record instead of a displacement record, and gives a reference to a paper in which a description of this procedure appears. Descriptions of such a device may also be found in the papers by M. A. Biot to which references were made at the end of the author's article. It is fortunate that more than one procedure is available since it allows a prospective user a choice of methods. The integration difficulty which Mr. White mentions has been largely overcome by the Coast and Geodetic Survey personnel who have made the necessary integrations for a number of quakes.

Mr. White objects to the title of the paper, principally it seems, because of the difficulty of applying the analysis to buildings and especially to that type of building which is constructed around a frame as the principal structural element. The question as to whether the type of analysis described will apply to a given structure must be decided upon the basis of its actual physical characteristics. If the stabilizing forces are provided principally by bending of beam elements, then the method will apply, otherwise not. It is the author's belief that, at present, the method will not apply to many buildings. This is because the frame is not the predominant element in so far as the rigidities opposing lateral deformations are concerned. It seems probable that estimates of frequency and modes of vibration for this important type of structure will have to be based largely on experimental data. The author can heartily agree that a combination of theory and experiment offers the greatest possibilities for further progress. It must be expected that data on damping will come almost wholly from experiment.

In returning to the theoretical aspects of the problem, Mr. White develops an expression for the response of a structure to ground motion based upon the idea of an elementary velocity increment. He is to be congratulated upon his choice of a starting point since it greatly simplifies the development.

Near the beginning of his discussion Mr. White identifies the principal sources of difficulty with the earthquake problem. These are the complex nature of the ground motions which must be dealt with, and the complexities introduced by distributed masses, resistances, and damping rates met with in nearly all structures. Present successes indicate that with some further improvements, torsion pendulum operation of some sort will yield an effective answer to the first difficulty. If some more expeditious experimental or computational procedures could be devised for finding the mode patterns with their associated moment and shear distributions, the second difficulty would be largely overcome.

In closing, the author wishes to thank those who have contributed to the discussions.

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Job Problems and Practice

WHAT JPP MEANS

—an informal clearing house for mutual help on concrete problems; an opportunity for the ACI member who *knows* to tell the ACI member who *needs* the information. If the subject matter holds probabilities of general interest it will be briefed in these pages—with authorship credit unless the contributor prefers not.

If you know of an interesting problem whose solution has smoothed the way of someone in the field, record it briefly. If you have a problem, present it; a question, ask it. If you are moved to helpful comment or constructive criticism write the Secretary.

The "answers" are those of individuals to whom the questions are referred and not of the ACI.

Causes and Control of Bleeding (38-93)

Q—What are the causes of bleeding and how can they be controlled?

By T. C. POWERS*

A—There are three aspects to the phenomenon of bleeding: one is the rate; another, the amount; and the third, the time for which bleeding continues. Our studies indicate that bleeding is neither more nor less than settlement, or sedimentation. The major factor controlling the rate of settlement is the amount of surface area in the mix in relation to the amount of water. The higher the water content or the lower the "surface content," the higher the rate and the greater the amount of bleeding.

The maximum possible amount of bleeding depends primarily on how far apart the particles in the mix are originally separated by water. In a mix in which the particles are not separated there can be no settlement; neither can there be plasticity. Therefore, any mix that is plastic has some capacity for bleeding.

The length of time that concrete can bleed depends first of all on the depth of the layer of concrete. A shallow layer can obviously become completely settled in a shorter time than a deep layer of the same materi-

*Assistant to the Director of Research, Portland Cement Assn., Chicago, at the 2nd Annual "Quiz Session"—38th Annual Convention.

al; but if the bed of concrete is deep enough, then a second factor enters: the length of time before the chemical processes cause the mass to congeal.

From what has been said, it is plain that the bleeding characteristics of concrete can be characterized completely only from a knowledge of three factors: the initial bleeding rate, the inherent capacity for bleeding, and the length of time during which the cement under the existing conditions will allow the concrete to bleed.

Bleeding can be controlled by increasing the "surface content" of the concrete. This can be done by adding pulverized stone; or sand containing a sufficient amount of subsieve material; or any fine mineral powder. The amount of any given material required to produce the desired effect cannot be predicted, for materials of apparently the same specific surface have different effects on bleeding. Bleeding can also be controlled with foaming agents.

It should *not* be assumed that any procedure that will reduce bleeding is desirable, for bleeding is not necessarily and not always harmful.

Reinforcing 12-in. Walls of Airplane Motor Test Building (38-94)

Q—Referring to "Building Regulations for Reinforced Concrete" (ACI 318-41) I happen to be particularly interested in concrete walls in connection with the construction of an airplane motor test building. The test cell walls are 90 ft. long, 19 ft. high and 12 in. thick. The load at base of the wall caused by wall and roof slab weight amounts to 35 psi.

Sections 1112 (a) and 1112 (i) of the ACI code indicate that the minimum wall reinforcement "in each direction, both vertical and horizontal" shall be "at least equal to .0025 times the cross-sectional area of the wall, if of bars". In the case at hand this would mean, .0025 times 144 divided by 2 equals .18 sq. inches, or, say ½-in. rds. 12 in. o.c., both ways and faces.

To me this seems to be too much steel and yet I hesitate to cut the amount down after reading the code. This is an important building and cracks would certainly be objectionable, yet I do not wish to be accused of wasting steel. Will you express an opinion as to whether the code regulations should be adhered to or modified in this case?

A—In the structure you describe, if the stress in the walls is due only to weight of the walls and the roof, that is to say, the walls will not be subject to bending resulting from inward or outward air pressure caused by the use of the building, I see no reason why the reinforcement could not be materially reduced or entirely eliminated. It is a common provision of building codes, such as that of the National Board of Fire Underwriters, the Pacific Coast Building Officials Conference and the Department of Commerce, to permit solid masonry walls which, of course,

include plain concrete 12 in. thick for walls as much as 35 ft. high. Such walls are required to be supported at right angles to the wall face at intervals not exceeding 18 to 20 times the wall thickness. In your case the walls being 19 ft. high and supported at top and bottom would conform to requirements for an unreinforced wall 12 in. thick.

Whether or not you decide to reduce or eliminate the reinforcement, I would recommend the use of control joints not to exceed 20 ft. if the walls are reinforced and 10 ft. apart if unreinforced. By providing control joints the location of cracks will be predetermined and can be made tight.

Use of Mud Jack in Strengthening Concrete Pavement (38-95)

Q—Should pumping a lean concrete mixture under pavements by mud-jack technique be a satisfactory method for strengthening pavements where necessary to carry military or other increased weight or traffic?

By JOHN W. POULTER*

A—From our observations, it would seem that for the most part pavement failures under traffic are progressive in nature. That is, a pavement may carry a large volume of heavy traffic for considerable time, often years, before any serious distress is evident; and further, that after distress is apparent, there is a progressive rate in the increase of the size of the area affected as well as the number of new areas appearing. There seems to be some evidence also that the rate at which distress progresses in a pavement goes up considerably faster than the increase in the volume of traffic. That is, a large number of heavy trucks following close together will do considerably more damage to a pavement than the same number of trucks passing over several days, all other conditions remaining the same.

Along with the affect of traffic weight, speed, and volume, there is the very definite relation in grade materials and drainage conditions.

For the most part, failure in a highway surface is always preceded by a grade failure, and in many cases, this base failure may precede the surface failure by months or even a year or more, as is particularly so with concrete pavement—the type referred to in this question.

With any rigid type of pavement, the affect of the action of traffic on the grade is more pronounced adjacent transverse joints and cracks, causing thin voids to be formed under the receiving end of the slab. This can be detected by the characteristic condition along the edge of the pavement long before the void has extended to the point where the slab is not sufficiently supported to carry the traffic. This condition

*Koehring Co., Milwaukee, Wisc., Mr. Poulter's reply was presented at the Institute's Second Annual "Quiz Session" at the 38th Annual Convention, Feb. 18-20, 1942.

has been successfully corrected by pumping in a rather fluid material that will set up and fill the void.

Careful inspection and the maintenance of uniform support under the pavement with a material having a texture somewhat similar to the rest of the grade would seem to be about all that could be done under the slab.

A lean concrete mixture would be very likely to leave many voids if it were forced under the slab, because of its harshness. This would give uneven support as well as an uneven effect on the grade due to the action of traffic. Also, there would be very little increase in the structural strength of the slab.

It would seem that where a pavement was required to carry much heavier traffic both as to volume, weight, and speed than it was designed for, it would be much more efficient to correct the grade condition both as to voids and drainage as much as possible and then add additional material to the surface to increase the weight of the slab, and, incidentally, the structural strength. More thickness and weight of slab is essential to carry increased weight and speed of traffic.

The ACI Code's Test Load Requirement (38-96)

Q—The ACI and other building codes require that test loads be equal to 1½ times the design live load plus ½ the dead load. Does this not penalize the heavier type of construction?

By F. E. RICHART*

A—This requirement applies to the superimposed test load; it is another way of saying that, including the weight of the structure, the total load for the test is 1½ times the total live and dead design load. Since the ACI Code uses the same working stresses for live and dead loads, it does not appear that these test loads impose any penalty on heavy construction. However, if one took the viewpoint that the capacity of the structure should be expressed as one design dead load plus some multiple of the design live load, it might be granted that the ACI test load is slightly unfavorable to a structure with a very high dead load. However, this conception is hardly consistent with the system of working stresses used, nor is it likely that a combination of very heavy dead loads and light live loads will often occur. As a matter of fact, 1½ design loads do not constitute a severe test load and will give relatively little information as to the strength of a structure, since the steel stresses particularly will be much less than the values computed in the conventional way, at this load. A structure that shows much distress at this load should certainly be subjected to a complete and careful investigation before it is placed in service.

*Research Professor of Engineering Materials, University of Illinois, Urbana.

Adopting the ACI Code by Reference (38-97)

Q—I should like information on the use of the "Building Regulations for Reinforced Concrete" (ACI 318-41) as a basis for a new county code. It would be desirable to draft the new code covering reinforced concrete merely by reference to the ACI Standard and subsequent revisions.

By J. P. THOMPSON*

A—Earlier editions of the ACI code have been used as a basis for the reinforced concrete sections of practically all municipal codes adopted within the last ten years or more. Some of the cities have practically copied the code verbatim, while some others have adopted it by reference. Among the latter are Cincinnati and Chicago. Cincinnati first adopted the ACI 1928 code and then adopted the 1936 edition when the latter was published. Adopting the code by reference rather than by complete publication makes it a very simple matter to keep the regulations up to date by a very short amendment to the general building code.

In adopting any code by reference a specific edition must be referred to and a phrase such as "and subsequent revisions thereof" can not be added. A phrase such as this would transfer legislative authority from the properly elected body to an outside source and consequently has always been ruled illegal. I believe that the only way of handling this situation so that the latest edition may be used as a guide without legislative action is to include a provision somewhat as follows:

The working stresses, design and construction of reinforced concrete construction shall conform to generally accepted good practice. For the purpose of this code, compliance with the "Building Regulations for Reinforced Concrete" prepared by the American Concrete Institute shall be considered as prima facie evidence of good practice.

Determining Cement Content of Hardened Concrete (38-98)

Q—How may the percentage of cement in hardened concrete be determined?

By ROY W. CARLSON†

A—There are two approximate methods of determining cement content of hardened concrete, both requiring chemical analysis. The first is to determine the percentage of soluble silica in the concrete and divide by 0.2175, on the assumption that the cement contains 21.75 per cent soluble silica and the aggregate contains none. If the silica content of the cement is known, the correct value naturally is used instead of .2175. This method is an A.S.T.M. Standard (C85-36).

*Structural Bureau, Portland Cement Association, Chicago.
†Mass. Institute of Technology, Cambridge, Mass.

The second method is to determine the calcium oxide content of the concrete and divide by the fractional part of lime assumed to be present in the cement. This method cannot be applied where the aggregates contain an appreciable or variable amount of lime, while the first method cannot be applied where the aggregates contain or develop soluble silica. In either case, a representative sample of the concrete is crushed and pulverized for the test.

Computing Stirrup Spacing in Non-rectangular Shear Diagrams (38-99)

By W. C. GOODWIN*

The code formula for stirrup spacing is simple and convenient when the shear diagram is rectangular, but because of the labor of successive shear deductions is seldom used directly when the diagram is triangular or trapezoidal. The convenience of the formula may be extended to triangular and trapezoidal diagrams by means of a little used relation between the computed spacing at any point and the distance of the point from the zero end of the shear triangle.†

Nomenclature

a = length in inches of base of shear triangle with altitude of $v - v_c$.

a' = distance in inches from the zero end of the shear triangle to a point where the stirrup spacing is s'.

a'' = distance in inches from the zero end of the shear triangle to a point where stirrup spacing is discontinued.

s, s' and s'' = stirrup spacings in inches at points distant a, a' and a'' respectively from the zero end of the shear triangle.

N = number of stirrups required in distance a for triangular diagrams and in distance $a - a''$ for trapezoidal diagrams.

A_v, b, d, f_v, j, w, V, v, v_c, have their usual meanings.

Introduction

In that portion of a prismatic beam carrying a uniform load of unvarying intensity (with or without concentrations) and reinforced with stirrups equally stressed, the product of the computed stirrup spacing s' at any point, by the distance a' from that point to the zero end of the corresponding shear triangle is constant. In Fig. 1, which shows that part of a shear diagram representing stress in the web reinforcement, $sa = s'a' = s''a''$.

The quantity sa is independent of v_c and is equal to $\frac{12\,A_v\,f_v\,jd}{w}$, which is constant so long as the beam is of constant section and the uniform load does not vary. The existence of vertical breaks (concentrated loads) in the shear diagram does not change sa.

*Wollaston, Mass.

†See Diagram 17, p. 81 in report of ACI Committee 317—"Reinforced Concrete Design Handbook."

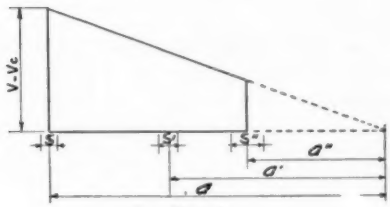

Fig. 1

Computation

(1) Compute the minimum spacing at the support,

$$s = \frac{A_v f_v}{b(v - v_c)}$$

(2) Compute the length of base of the shear triangle having altitude equal to $v - v_c$,

$$a = \frac{12V}{w} \cdot \frac{v - v_c}{v}$$

(3) Set the product sa on the slide rule.

(4) Read the spacing at any selected point,

$$s' = \frac{sa}{a'}$$

Example, Triangular Diagram

Compute spacings of ⅜-in. rod stirrups at one end of a beam where b is 12, w is 3000 lb., V is 30 kips, $v - v_c$ is 84, v is *144*, f_v is 16000.

$$s = \frac{.22 \times 16000}{12 \times 84} = 3.5$$

$$a = \frac{30}{3} \times \frac{84}{144} \times 12 = 70$$

$$sa = 3.5 \times 70 = 245$$

$a = 70$

1 @ 3 = 3 / 67) 245 = 3.7

3 @ 4 = 12 / 55) 245 = 4.5

2 @ 5 = 10 / 45) 245 = 5.5

2 @ 6 = 12 / 33) 245 = 7.4

1 @ 8 = 8 / 25) 245 = 9.8

1 @ 10 = 10 / 15) 245 = 16.3

$$N = \frac{a}{2s} = \frac{70}{2 \times 3.5} = 10, \text{ check.}$$

Example, Trapezoidal Diagram

Compute spacings of ½ in. rod stirrups at one end of a girder between the support and the first concentration 8 feet from the support. b is 14, w is 2000 lb., V is 36 kips, v is 134, $v - v_c$ is 74, f_v is 16000.

$$s = \frac{.40 \times 16000}{14 \times 74} = 6.2$$

$$a = \frac{36}{2} \times \frac{74}{134} \times 12 = 119 \quad (a'' = 119 - 96 = 23)$$

$sa = 6.2 \times 119 = 737$

		a
		119
3	@ 6	101
3	@ 8	77
2	@ 10	57
1	@ 12	45
1	@ 16	29
10		

$$N = \frac{(a - a'')(a + a'')}{2\,sa} = \frac{96 \times 142}{2 \times 737} = 9.3, \text{ check.}$$

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Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Standardization activities of national technical and trade associations

Division of Codes and Specifications, National Bureau of Standards, 288 p. 1942. Superintendent of Documents, Government Printing Office, Washington, D. C., 75 cents.

The work of 450 organizations, insofar as it is within the scope of the title, is described, listed. It is a directory of standardizing organizations (useful to those seeking the sources of authority in preparing specifications or other use of commercial, engineering and other standards.

Proportioning concrete materials by the unit layer method

J. C. Witt, *Rock Products*, V. 44, No. 12, Dec., 1941, p. 75. Reviewed by Roy N. Young

This method employs a common unit, called the unit layer, for all materials instead of the variety generally used such as sacks (of cement), gallons, cubic feet and cubic yards. This unit layer is a solid one-yard square and 0.01 yard thick. A chart on a scale practical for use is presented, and its application to the design of concrete mixtures is described.

Resistance of the Chicago area dolomites to freezing and thawing

H. B. Willman, *Rock Products*, V. 44, No. 12, Dec., 1941, p. 49-52. Reviewed by Roy N. Young

Samples were taken from dolomite formations representing about 350 of 450 ft. through the strata. The rocks were sorted into 15 types based on visible physical characteristics and insoluble residues. Results of tests indicate that resistance of these dolomites to freezing and thawing may be predicted with reasonable accuracy by their readily determinable physical and compositional properties. Coarser graining, higher macroporocity, lower insoluble residue were influences toward higher resistance to freezing and thawing.

Design and construction of tower silos

H. M. Pennington, *Concrete Building & Concrete Products*, V. 16, No. 10, Oct. 1941, p. 159-162. Reviewed by J. C. Pearson

Attention is called to the first installment of a continued article as a very promising summary of general silo information, not only with regard to the design itself, but also in the relation of silo capacity to the requirements for silage of various types of live stock. The subject is one of growing importance in Great Britain, where small silos of low height are more or less common, but where the author urges more consideration of the larger

and more economical types of high silos commonly used in the United States. The designs illustrated in this installment are confined to concrete stave silos. (Note: Conclusion of this article appears in the November issue.)

American Highway Practice, Volume I

LAWRENCE ILSLEY HEWES, 459 p. illus., John Wiley & Sons, Inc., $5.00 Reviewed by R. W. CRUM

The first volume of a comprehensive treatise by the Chief of the Western Region, Public Roads Administration covers the best current practice in the United States in highway location, roadway design and construction, highway landscape, sand-clay and stabilized roads, macadam roads, gravel and fine crushed rock roads, and intermediate bituminous surfaces.

A distinctive feature of the book is the extensive and well selected bibliography following each chapter.

Portland cement concrete pavement and other higher type surfaces are to be treated in a forthcoming Volume II.

Asbestos-cement tubes in roof construction

Concrete Building and Concrete Products, V. 16, No. 10, Oct. 1941, p. 163-165. Reviewed by J. C. PEARSON

Our English friends, perhaps as a result of war-imposed privations, seem to be progressing steadily in the direction of wider adaptation of concrete products to structural needs. This article describes a type of roof framing recently designed by two cement-asbestos companies in which the main members are cement-asbestos tubes formed into trusses with a very small amount of steel, and assembled by means of bolts and concrete connectors. Columns and main rafters are also of cement-asbestos tubes, and roofs are covered with cement-asbestos sheets. Ridges and gutters and other special shapes are formed in the same material. Tests of tubes as struts in various diameters and lengths have been made at the Building Research Station, and a table of working loads is given for different sizes.

Determining the surface area by air permeability method

S. L. MEYERS, *Rock Products*, V. 44, No. 12, Dec., 1941, p. 56-59. Reviewed by ROY N. YOUNG

This paper reports a study of the Lea and Nurse air permeability method of determination of fineness of powders as modified by Blaine, and comparisons are made between this and the Wagner turbidimeter method. The surface area results by the permeability method were decreased from 2954 to 2807 through a range of temperature from 65 to 95 F. No effect was found by varying humidity. The use of oxygen instead of air did not change the results, but hydrogen gave lower and natural gas gave higher results. This method gave higher surface area for cement after exposure to air than obtained on the unexposed cement. The ratio of surface area obtained by the permeability method to that by the Wagner method varied among a number of samples from 1.74 to 1.89, but the ratio showed no definite relation with fineness.

Dry process vs. wet process in portland cement manufacture

Cement & Lime Manufacture, V. 14, No. 12, Dec. 1941, p. 193-202. Reviewed by J. C. PEARSON

This subject, much discussed a decade or more ago, is reviewed at considerable length in this article, and takes into account changes made in the last few years in treatment of raw materials and methods of manufacture. The comparisons are summarized as follows: The dry process is in no way unsuitable for use when raw materials are normally dry and reasonably consistent in composition, and contain no constituents likely to prove harmful in the finished cement. Under comparable conditions, the dry process will consume about one-third less coal in burning the cement and will use less than one-fourth as

much water. The wet process alone is suitable for patchy or variable raw materials, and in conjunction with froth flotation or other separation process, is alone suitable for use when some constituent of the raw material has to be eliminated. The power required for either type of plant will be substantially the same.

Removing alkalis by heating with admixtures

HUBERT WOODS, *Rock Products*, V. 45, No. 2, Feb. 1942, p. 66-68. Reviewed by ROY N. YOUNG

Two dissimilar portland cement raw mixtures were given heat treatments in a gas-fired laboratory furnace to determine the effects of temperature and time of treatment upon the volatilization of the alkalis. Temperatures ranged from 1100 to 1350 C. and the durations from ½ to 2 hours. Other tests were made on charges containing admixtures. These charges were heated for one hour at either 1250 or 1350 C. In addition, some data from actual plant operations are given. The rate of volatilization of potash at 1100 C. was very low but was rapid at higher temperatures. The rate for soda was low at all temperatures. The proportions of potash and soda lost in the burning of cement raw mixes vary widely among plants but always the potash was volatilized more readily than the soda. More of both alkalis are lost in the dry process of manufacture than in the wet. It was concluded that: (1) Calcium sulfate retards volatilization of both potash and soda. (2) Calcium fluoride does not aid materially in volatilizing soda. (3) Sodium chloride has a large effect in expediting the volatilization of potash but actually increases the soda content materially. (4) Calcium chloride additions lower both soda and potash, but the effect appears to be much stronger on potash than on soda.

Conservation of critical materials approved by executive committee of AASHO

The Constructor, V. XXIV, No. 2, p. 39, Feb. 1942. HIGHWAY RESEARCH ABSTRACTS

Changes in design and construction standards for the conservation of critical materials have been approved by the Executive Committee of the American Association of State Highway Officials upon recommendation of the Committee on Standards. The announcement stated:

"Upon recommendation of the Committee on Standards of the association, the Executive Committee of the American Association of State Highway Officials, after consultation with the federal Office of Production Management, hereby promulgates changes in association design and construction standards due to the necessity for the conservation of critical materials during the national emergency.

"The objective of these changes is hoped to be attained by the adoption of designs which reduce or eliminate critical materials in construction projects by the substitution of non-critical materials wherever possible, and by simplification of standards and specifications to reduce the number of sizes and shapes of critical materials.

"The critical materials principally involved in road and bridge construction are copper, zinc, steel alloys, carbon steel and other iron products.

"The following changes in specifications for bridges and structures are recommended:

"1. Reduce the allowable unit working stress of concrete f_c to 700 lbs. per sq. in.; this working stress to apply to all classes of concrete which have an ultimate compressive strength of 2,400 lbs. per sq. in. or better. Some economy can be gained by reducing the amount of cement required in the richer mixes now used. An exception to this would be where it would increase the dead load to such an extent as to require a greater amount of steel, such as a concrete slab on a long steel span. 2. Use a value of n equals 10 for most concrete and eight if 4,000-lb. concrete is secured. 3. Use reinforced concrete in place of steel in bridge guardrails, I-beams, spans, etc. 4. Make full use of the composite action of concrete floors and beams by using mechanical anchors between floor and beams. 5. Use an allowable working stress for reinforcing steel of f_s equals 22,000

lbs. per sq. in. in culverts, abutments, piers, columns and practically all substructures, specifying intermediate grade steel. This would probably call for a more liberal use of waterproofing. 6. Replace heavy steel expansion dams with lighter steel design and concrete cross beams, where practicable. 7. Discourage the use of steel grid floors except in repair jobs and movable spans. 8. Eliminate the use of copper water stops, bronze expansion plates or any copper alloy. Sheet lead can be substituted in some cases. 9. Eliminate the use of aluminum paint and all galvanizing except perhaps galvanized cables and hardware in timber bridges. 10. Eliminate the use of all alloy steels, including silicon. 11. Eliminate the use of steel bearing piles for friction piles. 12. Make greater use of treated timber. 13. Substitute small concrete culverts or concrete or clay pipe in place of corrugated iron pipe culverts. 14. Avoid the addition of reinforcing steel to take compressive stresses in concrete members. 15. Repair existing structures, if practicable, rather than replace them with new structures.

"There is need for the reduction or elimination of steel from concrete pavements and for the substitution of masonry, unreinforced concrete, or concrete or clay pipe for galvanized metal culverts. The use of reinforcement in concrete pavement can be eliminated by the substitution of non-reinforced slabs for reinforced slabs. When such substitution is made it will be necessary to use slab lengths consistent with the design of non-reinforced concrete slabs. In slabs with thickened edges, dowels or other low transferring devices are required at transverse joints. By substituting pavement slabs of uniform thickness for slabs with thickened edges, a reduction of steel across joints may be made inasmuch as provision for load transfer is not required, and only enough steel is necessary across joints to maintain alignment. The elimination of galvanized metal culverts is for the purpose of conserving zinc and iron products. As substitutes for these pipe culverts, box culverts of equivalent capacity are recommended. Such culverts may be constructed with masonry or mass concrete side walls, and with mass or lightly reinforced concrete or arched tops depending upon span and available head rooms. Concrete or clay pipe may also be used. Wherever possible in designs calling for reinforced concrete, substitution of miscellaneous sizes of reinforcing bars instead of design sizes is recommended, in order to utilize such local or regional stocks as may be available, making in the design such redistribution of the steel as use of the miscellaneous sizes available may require. In all new designs involving unavoidable reinforcing, simplification of sizes of reinforcing steel is to be secured by using only ¼-in., ½-in., ¾-in., 1-in., and 1¼-in. deformed rounds. To conserve copper, it is further recommended that lighting projects be entirely eliminated. It is understood that the changes in design and practice resulting from the above suggestions and recommendations, when submitted in connection with federal-aid projects, will be acceptable as emergency standards and practice wherever they are in conflict with existing association standards."

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The Nature of Portland Cement Paints and Proposed Recommended Practice for Their Application to Concrete Surfaces*

Report of ACI Committee 616—Recommended Practice for the Application of Paint to Concrete Surfaces

R. E. COPELAND
Chairman

H. G. ARLT
T. H. CHISHOLM
HENRY A. GARDNER
R. L. HALLET
JOHN MARSHALL
T. J. PETERSON
JAMES T. RESIDE
WAYLAND W. RICE
E. W. SCRIPTURE, JR.
CLARA SENTEL
WILLIAM B. SINNICKSON
E. VIENS

INTRODUCTION

This report on portland cement paint is the first to be submitted by the committee. Study is also being given to the oil and synthetic resin types of paint and a report on their use is under consideration.

The painting of concrete has increased along with the increase of concrete structures of types for which some kind of decorative or water repellent treatment is considered appropriate or desirable. This widening utilization of paint has brought not only additional business to the paint producer and the painter but also additional problems and responsibilities. The permanence and annual cost of paint finishes on concrete are being viewed more critically. The consumer is beginning to demand more definite assurances and proof as to the suitability and service life of paints. He has learned from experience that some paint jobs last as long as 15 years, some not more than 2 years. He cannot afford to repaint his house or building every 2 or 3 years and wants to know just how the more durable jobs are obtained.

*Of the 13 members of Com. 616, 9 approved the present report; 2 disapproved and 2 refrained from voting. It is released by the ACI Standards Committee for publication here, as information and for discussion.

The need for more factual information is generally admitted. Some studies have been conducted and others are under way but the information now available or in sight is decidedly inadequate in many respects. It seems evident that the problem requires and merits more study than it has previously received.

Portland cement paint was developed specifically for application to concrete and masonry surfaces and is widely used for that purpose. Since portland cement and water are the principal constituents, the resulting paint film may be considered in much the same light as any thin layer of hardened portland cement. The following discussion and recommendations are based on this analogy and on observations in the field and laboratory, the latter including principally the investigative work conducted during recent years by the National Bureau of Standards and Portland Cement Association.

The report was submitted for review to a large number of portland cement paint manufacturers and was indorsed by a substantial majority of commenters from this group. Further criticism of the detailed recommendations during the present "proposed" status of the report will be welcomed by the committee.

GENERAL DISCUSSION

Definitions

The shorter term, "portland cement paint," will be employed instead of the longer expressions such as "portland cement base paint" or "portland cement-water paint." It is commonly understood that they mean the same thing and refer to water dilutable paints in which portland cement is the binder.

Paints that are manufactured and sold by paint companies and are ready for use except for the addition of and mixing with water will be called "commercial paint".

The term, "job mixed paint", will be applied to paints that are made up by the individual user from the basic constituents such as portland cement, etc.

Except where specifically noted the discussion and recommendations apply to both prepared and job mixed paints.

Recommended uses

Portland cement paints are suitable for use on exterior and interior portland cement concrete and stucco surfaces excepting floors or other areas subject to mechanical abrasion. Their use on other types of surfaces such as brick masonry, will not be discussed in this report. They are especially recommended where it is desired to decorate and reduce the water permeability of exterior walls built of porous, open textured

concrete such as concrete block masonry or where the concrete or stucco is damp at the time of painting or may become damp subsequently.

In general, cement paint films are of questionable value for stopping leakage through porous walls exposed to appreciable water pressure, particularly if the paint is applied to the side away from the water pressure. Better results will be obtained if the side exposed to the water is painted. Two or three heavy coats of paint are sometimes successfully employed on the inside surface of the basement walls of houses to stop slight leakage or dampness. Severe conditions usually require more positive waterproofing methods.

The resistance of such paints to aggressive solutions and fumes is not significantly different from that of a good quality concrete and because of their thinness they will provide only temporary protection when exposed to severe corrosive conditions. However they are useful for protecting concrete of poor quality where the exposure is not too severe and it is feasible to repaint as often as may be required.

General characteristics

The typical portland cement paint film is hard, strong and relatively brittle. This latter property together with drying shrinkage sometimes results in the development of fine crazing or checking. Exterior films seem to be more susceptible to this trouble than films which are protected from the weather.

Such films generally adhere satisfactorily to the concrete and between coats and seldom fail by peeling or blistering. The occurrence of these defects almost invariably is due to inadequate preparation of the surface.

The typical film possesses good decorative qualities as to hiding power and color. However, when wetted, as by rain, it becomes slightly translucent or darker in shade and appears somewhat less satisfactory than when dry. However, on drying out, the film returns to its original color and opaqueness. Tinted paint films also assume a deeper shade of color when wetted.

Durability depends on several factors including the composition of the paint, texture and cleanness of the concrete surface, application procedure and the severity of the exposure. Based on information from several sources it appears that portland cement paints properly applied to exterior wall surfaces exposed to the weather may be expected to have a service life of about 8 years. Some paint jobs last considerably longer; some are less durable. The less durable jobs frequently are those involving smooth, dense concrete surfaces where there is the tendency to brush out the paint too thin and to neglect the requirements for satisfactory adhesion.

By service life is meant the time required for the paint to depreciate physically to a point at which it no longer provides adequate protection or coverage. According to this concept it is not determined by a reduction in decorative value except, of course, when poor appearance results from physical deterioration. All paints suffer in appearance from the normal accumulation of dirt and soot stains. Efflorescence usually comes from the concrete rather than the paint and is particularly noticeable if the paint is tinted. However, these and similar discolorations are not construed herein as terminating the serviceable life of the paint despite their effect on appearance.

Opinions differ as to whether portland cement paints are especially susceptible to soiling and staining due to their greater porosity as compared to oil and resin bound films. This matter has been under observation for some time in paint studies being conducted by the Portland Cement Association. In approximately 500 specimens exposed from 18 months to 5 years to rather bad sooting conditions the organic paints, on the average, became soiled at about the same rate as the portland cement paints.

Perhaps one of the most important qualities of portland cement paint is its tolerance to the moisture and alkaline conditions to which coatings on concrete are frequently subjected. Prolonged saturation or alternate wetting and drying or freezing and thawing of the concrete subsurface constitute conditions harmful to any paint film and are perhaps more successfully withstood by coatings of the portland cement type.

It is logical that paints on interior surfaces or other areas protected from the weather should last longer than when applied to exterior surfaces. Frequent cleaning however, may markedly erode the film and shorten its life.

DURABILITY FACTORS: CLIMATE, COMPOSITION, PROCEDURE

Climate is generally conceded to be a factor in the performance of exterior coatings. However, a properly formulated and applied paint should give satisfactory service under all climatic conditions within the range experienced in the United States and this factor seldom if ever excuses an early failure.

In seeking the reasons for poor paint performances there frequently is the tendency to overstress one possibility and to neglect the others. The kind of failure oftentimes gives a clue as to the primary cause.

Peeling or flaking is the result of poor adhesion, usually between the paint and concrete. This property depends primarily on the condition of the concrete surface when the paint is applied. Portland cement paint will not adhere properly to surfaces that are glassy smooth or are soiled or coated with foreign matter. Such surfaces must be well cleaned and

if necessary, acid washed or sand blasted to open the pores and provide suction and "tooth" for the paint. A weak bond, but less seldom peeling, results if the concrete is not dampened prior to painting and the paint film is allowed to dry rapidly. It appears that adhesion is influenced more by differences in painting procedure than in composition although there is some evidence that flaking may be caused if the paint contains an excessive amount of unhydrated lime or water repellent. With this exception failures of this type usually can be traced to some negligence or oversight in the procedure.

Rapid chalking is a characteristic quality of soft, weak films and leads to proportionately rapid deterioration in the color, opacity and durability of the coating. Usually it is due either to a deficiency of portland cement in the paint ot to failure to provide the damp curing necessary to the proper hydration and hardening of the cement. Soft films also may be caused by ingredients which prevent normal reactions between the cement and water.

As previously mentioned, portland cement paints very rarely fail by blistering and little has been learned as to its causes from the few cases studied. The presence of excess or free water on the surface being painted or the use of too much water repellent in the paint have been suggested but not definitely verified as possible causes.

Poor opacity and color value may be due to some deficiency in the paint composition, to the use of too much water in the paint mix, to applying the paint too thin or to the continued presence of excessive moisture behind the film.

The appearance of tinted paints will be marred if efflorescent salts are deposited on the film surface. For this reason tinted paints should not be applied to concrete that is actively effloresc ing or is in direct contact with water containing sulphates or other salts which might permeate to the painted surface. In the latter case it frequently is feasible to build the structure so that extraneous water will be excluded from the areas to be painted.

In general, it appears that much of the efflorescence trouble is caused from painting concrete at too early an age while it is still emitting appreciable excess moisture. In the case of tinted paints it is good practice to postpone painting as long as feasible.

A few pinholes in the film are not objectionable except when the paint is depended upon to reduce effectively the permeability of the surface. They result when the paint is not worked into the surface voids or when entrapped air breaks through the fresh film where it bridges instead of fills the voids. Pinholes can be eliminated by vigorous brushing. Composition is a slight factor in causing this defect in so far as it affects the workability and surface tension of the paint. Sprayed coatings usually

contain more pinholes than brush applied films especially when the receiving surface is coarse textured or contains many deep pits or voids.

The fine crazing observed in paint films of this type is logically related to the shrinkage, tensile strength, extensibility and other physical properties of the film which in turn vary with the composition of the paint. However, the observed differences in crazing due to the ordinary variations in composition are not as great as might be supposed.

Serious crazing, involving fractures extending through the paint film and more than 0.002 in. wide, usually is related to and induced by the development of crazing of the concrete surface subsequent to painting. Crazing over mortar joints also is usually traceable to fine shrinkage cracks in the mortar.

Normal crazing is not progressive, generally being completed within a few weeks after starting and in itself is not a serious defect. Observations by the Portland Cement Association on several hundred test specimens over a period of 4 years showed no significant change in the extent of crazing after the first year.

DISCUSSION OF COMPOSITION

Considering the nature of portland cement paint it appears that durability is dependent upon the strength, hardness and density of the paint film. Excluding the effect of procedure, it also is logical that these properties are controlled primarily by the portland cement content and secondarily by the inherent durability of the other constituents and their effect on the hardening of the cement-water mixture.

It also can be accepted as a basic fact that composition influences the appearance or decorative quality of the paint.

These remarks are merely to establish the importance of composition as a factor of paint performance independent of painting procedure. A paint that is deficient in those ingredients essential to good durability or appearance will invariably produce relatively unsatisfactory results no matter how carefully it is applied.

It is desirable therefore in this discussion, to introduce certain more or less fundamental concepts concerning the functions and value of the various constituents commonly used in portland cement paints. From such an analysis and the available test data and service records, it is possible and appropriate to indicate suggested percentage limits for most of the constituents.

Commercial compositions

Based on extensive chemical analyses, it appears that commercial portland cement paints range in composition about as follows:

Ingredient	Per Cent by Wt.		
	Min.	Max.	Typical White Paint
White portland cement	20	91	61
Hydrated lime	0	60	30
Inert fillers, (fine silica, limestone, etc.)	0	40	
Hygroscopic salts, (calcium or sodium chloride)	0	7	3
Water repellents, (stearates)	0	1.5	0.3 (or less)
Opaque pigments (zinc sulfide or titanium dioxide)	0	10	3
Water and organic matter (principally casein)	0	14	2.5 (or less)
Color pigments	0	8	

One or two commercial paints contain coarse inert filler consisting of white silica sand (No. 100—50 mesh fraction). Such paints, by virtue of the coarser particles, produce relatively thick films having a characteristic "sandy" texture.

Compositional requirements of federal specifications

The recently promulgated Federal Specification for Paint; Cement-Water, Powder, White and Tints (For Interior and Exterior Use), Designation TT-P-21* provides for two types of portland cement paint with compositional limits as indicated in Table 1.

TABLE 1

Ingredient	Per Cent by Weight			
	Type 1 Class A		Type 2 Class A	
	Max.	Min.	Max.	Min.
Portland cement	—	65	—	80
Hydrated lime	25	—	10	—
Water repellents (calcium or aluminum stearate)	1	0.5	1	0.5
Hygroscopic salts (calcium or sodium chloride)	5	3	5	3
Titanium dioxide (TiO_2) Zinc sulfide (ZnS) or mixture	5	3	5	3

Other provisions and requirements pertinent to this discussion are:

(1) The paint shall contain no organic binder.

(2) Color pigments shall be limeproof and shall be incorporated by a milling operation.

(3) The total free (unhydrated) calcium oxide (CaO) and magnesium oxide (MgO) in the hydrated lime shall not exceed 8 per cent by weight of the hydrated lime.

(4) Carbonates (calculated as carbon dioxide CO_2) shall not exceed 3 per cent by weight of the paint powder.

*Obtainable from the Superintendent of Documents, Washington, D. C., price 5 cents.

(5) The inclusion of silicious aggregate (No. 100 to No. 20 sieve) with either of the above compositions in the proportions of 15 to 40 lb. of aggregate to 60 lb of the basic paint powder is permitted and is suggested in the case of paint to be applied to open texture surfaces such as lightweight aggregate block.

It is to be noted that the typical commercial product conforms closely as to composition to the Type 1, Class A paint. The specification indicates no preference between Type 1 and Type 2 except to say that the latter should be used where there is excessive moisture as in water tanks and swimming pools.

Portland cement

Tests and experience indicate that the most durable coatings of this type are those in which portland cement comprises a high percentage of the minus No. 100 mesh material. In the Portland Cement Association tests, straight white portland cement films show negligible erosion and are in excellent condition after 5 years of weather exposure. Replacing 50 per cent of the cement by weight with hydrated lime or fine inert filler adversely affected the hardness and weather resistance of the film. Coatings made without portland cement but with lime as the binder were softer and more rapidly eroded than those containing cement. These indications of the importance of the cement content with regard to paint durability are in accord with experience in the field and the concensus of competent opinion expressed to the committee. Based on these considerations it is recommended that where a paint service life of 5 years or more is desired, portland cement should comprise not less than 65 per cent of that portion of the paint powder that passes the No. 100 sieve. For maximum durability or for coatings to be exposed to excessive moisture as in swimming pools it appears that this value might well be increased to 85 or 90 per cent.

Hydrated lime

The effect of hydrated lime as a constituent of cement paint may be summarized as follows:

1. The addition of 10 to 15 per cent of lime by weight enhances slightly the whiteness and hiding power of the dry film but may reduce the opacity of the wet film. The effect is not great and depends on the quality and fineness of the lime and cement. A small amount of opaque white pigment is more effective than lime for this purpose.

2. Workability and particle suspension of the paint mix may be improved depending on the plasticity of the lime used.

3. In the opinion of some investigators, the use of a small amount of lime seems to increase the miscibility of some water repellents such as calcium stearate, thus shortening the time for proper mixing.

4. Experimental data on the effect of lime content on the crazing tendency of paint films are very inconclusive except to indicate that the effect is not significant one way or the other.

5. Depending somewhat on atmospheric conditions there is rather rapid and thorough carbonation of the lime contained in thin coatings and some fairly durable paints containing as much as 50 per cent lime have been produced. However, with due consideration to such examples, it seems evident from what is now known about paints of this type that the replacement of an appreciable percentage of the portland cement with lime tends to impair the weather resistance and service life of the paint.

It thus appears that although lime is not an essential ingredient it probably may be safely used up to 25 per cent of the portland cement content. While it cannot be said definitely that the 25 per cent value should never be exceeded, the possible detrimental effects of higher lime content on the durability of exterior films seem to be more obvious than any benefits that might accrue.

Fine fillers (extenders, plasticizers, etc.)

The value of finely divided inert fillers such as pulverized silica, limestone, etc., is somewhat obscure, whereas when used in appreciable percentages they have been observed to cause excessive chalking and resulting therefrom, unsatisfactory durability and appearance with continued exposure. Being of about the same fineness as the portland cement they dilute and weaken the cement matrix with proportionate consequences on the weathering qualities of the film. Even though such fillers may reduce shrinkage somewhat or enhance slightly such properties as brushability it appears that their overall effect is more likely to be detrimental than beneficial.

Small amounts of colloidal material such as plastic clay, etc. are sometimes added to improve the workability and suspension qualities of the paint mixture but their effect on the durability of thin cement coatings is not very well known. Probably they should be avoided or at least restricted to small amounts (2 or 3 per cent) until more thoroughly tested as a paint ingredient.

The use of organic compounds for better dispersion and workability is not recommended except where they have been thoroughly investigated and shown not to be detrimental to the paint film.

Coarse fillers (aggregates)

Favorable results have been obtained with relatively coarse fillers, usually a siliceous material of acceptable color and weathering qualities such as fine white silica sand. In general the softer sands such as those derived from limestone and marble have proved less satisfactory as paint aggregate. The sand should pass the No. 20 sieve with not less than

90 per cent retained on the No. 100 sieve; the No. 100 to No. 50 mesh fraction is perhaps the most satisfactory for this purpose. Because it is much coarser than the cement, such filler does not weaken the cement matrix to the same extent as do the very fine fillers. The sand may be used in varying amounts but not exceeding equal proportions with the portland cement. Based on experiences on several jobs and on 2 to 4 year exposures of painted concrete masonry specimens included in a current Portland Cement Association investigation such filler appears to improve the surface hardness and durability of the paint. The sandy texture of the paint is considered attractive by many persons and it insures excellent adhesion between successive coats.

Mixtures containing sand cannot be applied with a hair brush but readily go on either smooth or rough textured concrete if a stiff bristle brush is used. The use of aggregate introduces a minor inconvenience in requiring more frequent stirring of the mixture in the paint bucket in order to keep the sand in suspension.

It is not recommended that aggregate be made a required constituent of portland cement paints but its indicated value in paints that are to be applied to concrete masonry or other coarse textured concrete should not be overlooked.

Hygroscopic salts

There are used as curing aids; presumably they help keep the cement moist for several days with water drawn from the air through their efficacy in this respect depends on several factors and should be more specifically evaluated. Sodium chloride (common salt) tends to weaken the cement and increase the time of initial and final set whereas calcium chloride accelerates the rate of hardening and in amounts up to 4 or 5 per cent has no significant effect on strength. Thus limited, neither salt appears to be harmful as to the shrinkage, weather resistance, opacity or other properties of the paint film. Such salts are a possible source of efflorescence but no evidence has been presented indicating that it is of a serious nature. Pending the development of additional data their use in amounts not exceeding 5 per cent is recommended. Calcium chloride is the preferred material because of its greater hygroscopicity and its accelerating action which is desirable for reducing the curing period and time during which the paint, in an unhardened state, might be damaged by rain or freezing weather.

Water repellents

Many of the prepared paints contain water repellents, usually either aluminum or calcium stearate powders ground with the base. Sometimes paste forms of stearate are stirred into the mixing water and added on the job. If powdered stearate is added on the job it must be very thor-

oughly mixed with the white portland cement and other ingredients prior to adding the mixing water; in such cases it is preferable and more convenient to use a waterproofed cement in which the stearate has been intimately mixed by a grinding operation.

The comparative merits and optimum amounts of the different water repellents as regards some properties of portland cement paint, particularly durability and shrinkage, are not well known. Some tests have been made on paints containing stearates incorporated both by grinding and by hand mixing and in addition there are some data on the effect of these substances on the strength and absorption of portland cement mortars. These data indicate that the amount of chemically pure stearate in the paint should range between 0.5 to 1.0 per cent of the portland cement content if water repellency and low absorption are to be obtained without too great a reduction in strength. However some authorities contend that 0.3 to 0.4 per cent of stearate is sufficient when it is ground with the paint and therefore is more homogeneously incorporated than is possible by hand mixing. Tests have shown that 2 per cent of calcium stearate ground with the base seriously reduces the strength and hardness of the paint film but that 1 per cent can be safely used. It also was tentatively indicated that 1 per cent of calcium stearate appreciably reduced the amount of crazing but 0.5 per cent or less was ineffective in this respect. Water repellents tend to add somewhat to the workability of the paint and the opacity of the wet film. Their effect on weather resistance has not been specifically determined as yet but it should be favorable. Proper damp curing and pre-wetting of the receiving surface are especially important when applying paints which contain water repellents because once dried they are difficult to re-wet.

It thus appears that small amounts of water repellents are not detrimental to the paint and offer certain advantages which may well justify their use. Amounts less than 1.0 per cent by weight of the portland cement content is recommended. It is expected that tests now under way will throw more light on this constituent.

Opaque pigments

White paints should contain a small amount of opaque white pigment such as titanium dioxide or zinc sulfide for the purpose of increasing the whiteness and hiding power of the film. Amounts of the commercially pure pigment of from 2 to 5 per cent by weight of the portland cement content are sufficient and can be safely used. Due to their fineness it is possible that appreciably larger amounts might be detrimental as to shrinkage, crazing and strength. Opaque white pigments usually are not used in tinted paints although it seems that they might be worth while in the case of light colors such as cream and buff.

Organic substances (casein)

Casein or other organic binder are contained in some portland cement paints presumably to improve workability and the adhesion of the paint to non-porous surfaces. Even small amounts of such material may prevent the proper hardening of the cement and thus seriously impair the durability of the coating and their use in portland cement paints intended for use on exterior concrete or stucco surfaces is not to be recommended.

Color pigments

Color pigments should be commercially pure oxides, very finely ground and lime proof. There are different commercial grades varying as to quality and price. Only the best pigments, with high tinting strength, should be used in paint. The amount of pigment required to give the color value desired depends upon its tinting strength and dispersion in the paint. For best results tinted paints should be produced in a grinding mill.

Water

The amount of water required to reduce the dry material to painting consistency varies for different paints and depends principally upon the fineness of the materials. The mixed paint should have the consistency of a fairly thick cream. Too much water results in a thin film and reduced opacity and durability. Although it is entirely possible to mix the paint too thick it appears to be the more common tendency to make it too thin.

Manufacture of paint

To produce a paint powder of uniform composition and quality the materials for each batch must be accurately proportioned by weight and thoroughly mixed together.

In the manufacture of commercial paints, the mixture of finely divided ingredients generally is reground in a suitable ball or pebble mill. This grinding operation results in the better dispersion of the ingredients and a finer and more uniform product than can be obtained by cruder mixing methods. Finer grinding also enhances the suspension and brushing qualities of the paint.

Tinting pigment should be proportioned to an accuracy of 1 per cent and milled in with the base. In view of these requirements it is recommended that only tinted paints of commercial manufacture be used.

Powder forms of stearate (aluminum or calcium stearate) preferably should be milled in with the paint powder or the cement.

For white paints the minimum mixing considered essential to satisfactory results consists of thoroughly stirring the ingredients together in a suitable container, then sifting this mixture through a sieve of No. 14 mesh fly screen. Batches of 100 lb. or more may be mixed in a small stucco or mortar mixer.

Paste water repellents, when used, should be mixed with the gauging water on the job. Calcium or sodium chloride if added on the job, also should be incorporated with the gauging water in the proper proportions. In commercial paints such salts are included as a part of the paint powder.

Aggregate, when used, should be dry mixed with the paint powder before the gauging water is added.

Paints must be properly protected from the atmosphere while in storage. Experience indicates that those that contain hygroscopic salts should be kept in air tight containers.

While satisfactory results have been and are being obtained with paints that are prepared on the job, the convenience and advantages of using a properly formulated ready-mixed paint should be obvious.

DISCUSSION OF PAINTING PROCEDURE

(a) Age of concrete

In general, it is desirable to defer painting until the portland cement stucco or concrete, including concrete masonry, is at least 24 days old. The application of tinted paints to cast-in-place concrete walls should be postponed longer, several months after the curing period if possible, so that the concrete will be drier and a less likely source of efflorescence. Other advantages of permitting cast-in-place concrete to age longer before applying either white or tinted paints are: (a) weather tends to overcome the effects of form oil; (b) the suction of the surface is more uniform; (c) the paint film usually crazes less and also tends to cover any such checking or crazing which may develop in the concrete surface itself prior to being painted.

(b) Preparation of surface

"The minimum amount of surface preparation should consist of thoroughly removing all dirt, dust, form oil and efflorescence from the surface. Dirt and dust should be washed off with clean water, using scrub brushes if necessary. Soaps or other organic cleaners should be avoided because unless completely removed they may prevent proper adhesion of the paint. To remove efflorescence first wet the concrete, then apply a 20 per cent solution of muriatic acid and after about 5 minutes scour off the efflorescence with stiff bristle brushes. If necessary, a stronger solution may be used. It is best to work on small areas, not more than a few square feet. The surface must be thoroughly washed with clean water after an acid treatment. It is practicable to remove traces of form oil with steel brushes, abrasive stones or lye solutions. However, if the surface is generally coated with oil, it will be more effective and economical to sandblast lightly the entire area to be painted or to postpone painting until the oil has been removed by weather action which, as previously indicated, may require several months.

Some concrete surfaces are so dense (glazed) as to make good paint adhesion very difficult to obtain. Concrete that is cast against plywood, Presdwood or steel forms frequently presents such a problem. Before being painted such surfaces should be acid washed, lightly sandblasted or dry rubbed with coarse grit abrasive stones, until the glaze is removed and positive "tooth" and suction for the paint are provided.

Old coatings of the organic type must be completely removed before applying portland cement paint. Usually this can be done most effectively and economically by sandblasting.

(c) Dampening concrete

Before applying the paint the concrete should be thoroughly wetted to control surface suction and to provide a reserve of moisture to aid in the proper hardening of the paint. A garden hose adjusted to give a fine spray is well suited for this purpose. A superficial dampening with a brush dipped into a bucket of water, as is so often done, is wholly inadequate.

Usually it is sufficient to wet concrete masonry walls or other concrete that readily absorbs moisture in one operation not more than one hour before painting. The spray should be applied in such manner that each part is sprayed 3 or 4 times for about 5 to 10 seconds each, depending upon the absorptivity of the surface, time being allowed between applications for the moisture to soak into the concrete; the unabsorbed water serves no purpose. If the surface tends to dry rapidly, as it may in hot weather, it should be redampened slightly just in advance of painting; it should be moist but not dripping wet or with a noticeable water film when the paint is applied.

Dense concrete absorbs moisture so slowly that it should be wetted in at least two operations not less than 30 minutes apart. It will be more effective to dampen rather large areas in advance of painting so that ample time is allowed for the moisture to soak into the concrete. When the paint is applied the concrete surface should be moist but without free water; some suction is necessary.

(d) Preparation of paint

Portland cement paints are made ready for use by thoroughly mixing the paint powder and aggregate, if used, with water in the proper proportions.

The proper amount of water will vary depending upon the fineness of the dry materials and should be determined by trial. The mixed paint should have the consistency of rich cream except that a slightly thinner consistency is recommended for the first coat applied to open textured types of concrete surfaces such as concrete masonry units.

For trial batches the following proportions of water to dry material are suggested:

Paint, Dry Material, 10 lb.	Water, Qts.	
	For Standard (Rich Cream) Consistency	For Medium Thin Consistency (First Coat on Open Textured Concrete)
Commercial paints (constituents mixed and finely reground	3.25 to 4.25	3.75 to 4.75
Paints, not reground, less than 1 per cent retained on No. 100 sieve	2.75 to 3.25	3.25 to 3.75
Paints, not reground, containing coarse filler; more than 25 per cent retained on No. 100 sieve	2 to 2.5	2.5 to 3

Mixing procedures vary but in general it is good practice first to reduce the dry material to a stiff paste with about half the estimated amount of water, after which additional water should be gradually stirred into the paste until the desired consistency is obtained. When the proper proportions have been determined it is advisable to make all batches the same except when a change in surface texture or other conditions requires a different consistency.

The paint should be vigorously stirred for several minutes until it is uniform in color and consistency and all particles are thoroughly wetted. Workability will be improved by allowing the mixture to soak or prehydrate for 30 to 45 minutes prior to application. Most paints remain in usable condition for 3 to 4 hours after being prepared although in hot weather some paints, especially those containing calcium chloride, should be used somewhat sooner.

While being used the paint usually tends to stiffen slowly due to chemical and physical reactions and evaporation of the water, and it is common practice to thin the mixture with additional water when necessary to maintain the desired consistency. There seems to be no serious objection to retempering white paints provided it is properly done. The paint should first be vigorously restirred as sometimes this operation will restore its fluidity. Any additional water required should be added and thoroughly incorporated in small increments.

In the case of tinted paints it is advisable to avoid the necessity of retempering in so far as possible as the color may be altered slightly if the proportions are appreciably changed. Retempering can be largely avoided by making smaller batches which will be used before the paint stiffens excessively or by keeping the mixed batch in a tightly covered container from which the painters' buckets are filled.

(e) Application of paint

The surface should be uniformly damp but with no free water.

For obvious reasons, no painting should be done on frozen concrete or when the paint may be exposed to temperatures below 40 F. within 48 hours after application. Some paint manufacturers prefer a minimum temperature of 50 F.

The paint should be applied in two coats of the same color. Preferably not less than 24 hours should be allowed between coats; in no case should the second coat be started until the first coat has become sufficiently hard to resist marking by the brush being used. In hot, dry weather, the first coat should be slightly moistened before applying the second coat. "Doubling back" or the application of a double thick coat is not conducive to good results and should not be permitted. Three-coat work may be employed if desired but the extra coat should not be necessary and is seldom worth the additional cost.

Most portland cement paints cannot be satisfactorily applied with the ordinary hair bristle paint brush. Proper application to coarse textured concrete requires a brush with relatively short, stiff fiber bristles such as fender brushes (commercially sold for cleaning the underside of automobile fenders) or ordinary scrub brushes. If the concrete is of smooth or sandy finish, white wash or dutch type calcimine brushes are suitable. The paint should be brushed on in such manner as to be uniform in thickness and free from "ridging" or excessively heavy brush marks.

It is true that some commercial portland cement paints exhibit smooth working qualities more on the order of oil vehicle paints. However, experiments indicate that even these paints should be applied with stiff bristle brushes if the concrete is coarse textured.

When weather conditions are such as to cause the paint to dry rapidly it is advisable to work "in the shade" in so far as practicable. Such practice makes it easier to keep the surface uniformly moist for proper suction and helps prevent too rapid drying of the coat being applied.

The different constituents of portland cement paint vary substantially in specific gravity with each other and with the water; consequently to prevent segregation and maintain a uniform mixture, the paint should be stirred frequently in the bucket.

When painting concrete masonry or other porous concrete for the dual purposes of "moisture proofing" and decoration, both coats should he vigorously scrubbed on in such manner as to work the paint back into the voids and provide a continuous paint film free from pinholes or other openings through which water might penetrate. Tests of the rain resistance of painted concrete masonry walls* indicate that paints

*Tests of the Resistance to Rain Penetration of Walls Built of Masonry and Concrete—R. E. Copeland and C. C. Carlson; ACI JOURNAL *Proceedings*, Nov. 1939; V. 36, p. 169.

applied with spray equipment provide less protection than scrubbed on coatings. Spray application therefore is recommended only on dense concrete or interior surfaces or where the paint is not required for waterproofing purposes.

While excessively thick films are to be guarded against there seems to be a tendency in practice to use too much water in the paint and to brush it out too thin. Such coatings may look well at first but generally lose their opacity and protective value much sooner than thicker films.

The proper average film thickness and coverage rate is rather difficult to estimate for portland cement paints because of the effect of differences in the texture of the concrete to be covered. However, considering smooth concrete surfaces, it appears that a two-coat film should average about 0.015 in. thick which will be obtained with a coverage rate of about 100 sq. ft. per gallon of mixed paint for the two coats. If the concrete is rough textured or if the paint contains coarse filler (No. 50 to 100 mesh) a film thickness of 0.025 to 0.035 in. and coverage rate of from 45 to 55 sq. ft. per gallon should be sought. Two coats on very rough textured block may require as much as one gallon per 35 sq. ft. of surface. It is questionable economy to stretch the paint much further than these figures indicate. Laps, of course, should be well brushed out.

There is perhaps less objection to the use of thinner films when repainting provided the new and old paints are of the same color and the old paint is in fair mechanical condition.

(f) Curing

Proper hardening of paint films of this type depends upon the availability of moisture for chemical reaction with the portland cement. The moisture in the concrete base, in the paint itself and in the air is utilized for this purpose but usually this is not enough. On most jobs it is practicable to sprinkle the painted surfaces two or three times a day with the same fog spray used for dampening the concrete and it is recommended that this be done between coats and for at least two days for the final coat, starting just as soon as the paint has hardened sufficiently not to be damaged by the spray, usually about 12 hours after application. Damp curing in this or some other effective manner must be provided in order to obtain the best results with portland cement paints. It will improve the hardness and durability of the paint in every case and in some instances will mean the difference between a satisfactory and a poor paint job.

Discussion of this report should reach the ACI Secretary in triplicate by Aug. 1, 1942.

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The Properties of Lightweight Structural Concrete Made with Waylite Aggregate*

BY GEORGE W. WASHA†
Member American Concrete Institute
AND KURT F. WENDT‡

SYNOPSIS

Mix design methods and procedures for securing fluid workable concrete without bleeding have been developed. Strength, absorption, specific weight, toughness, stiffness, and durability characteristics of structural Waylite concrete are presented. Damage due to freezing and thawing was predicted from the change in the dynamic modulus of elasticity with excellent results. Reinforced concrete beams made with Waylite aggregate closely followed the behavior observed for sand-and-gravel concrete beams.

INTRODUCTION

The need for information regarding the mechanical properties and mix design of lightweight concrete, because of its ever-increasing applications in the fields of pre-cast construction and job-placed reinforced structures, prompted several studies at the University of Wisconsin. Aggregates such as Pottsco, cinders, Haydite, and Waylite were included in brief initial studies. Because of wide-spread availability and almost complete lack of information in comparison with that for cinders and Haydite, interest centered on the lightweight slag, Waylite, and the major work was done with this aggregate. The Waylite Co., Chicago, Ill., producers at several locations in the East, South, and Midwest, financed a considerable part of the later studies.

Transverse strengths and specific weights for Series A were obtained from tests of 102 prisms, 2 by 2 by 9 in., compressive strengths from modified cube compression specimens, and absorptions from beam fragments. Moduli of elasticity were obtained from tests of 15 cylinders, 6 by 18 in.

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†Instructor, Dept. of Mechanics, University of Wisconsin.
‡Associate Professor, Dept. of Mechanics, University of Wisconsin.

In Series B, load-deflection curves were obtained for 36 beams, 3 by 4 by 39 in. tested on a 36-in. span from which the values for moduli of elasticity, moduli of rupture, and energy of rupture were obtained. Compressive strengths were obtained by the modified cube method. Freezing and thawing tests were made on 48 beams, 3 by 4 by 16 in. The damaging effects of the freezing and thawing action were measured by the dynamic method.

The characteristics of reinforced concrete beams made with Waylite aggregate were checked against those made with Janesville sand and gravel by testing 8 beams, 6 by 12 by 114 in. on a 9 ft. span. In these tests stresses calculated from the usual formulas were compared with those calculated from actual strains. Compressive strains in the concrete, tensile strains in the longitudinal steel and in the stirrups, and end slip of the longitudinal reinforcing bars were read at frequent intervals during each test.

AGGREGATE PROPERTIES

Waylite is a steam-centrifuged, inflated, blast-furnace slag, cellular in structure. The fine material, graded from No. 4 sieve to dust, weighs from 55 to 65 lb. per cu. ft. on a loose, dry basis; coarse material from No. 4 to ⅜ in. weighs 35 to 40 lb. per cu. ft. On the same basis, Janesville gravel weighs about 90 lb., Janesville sand 105 lb. and University sand 98 lb. The mechanical analyses of the aggregates used are given in Table 1.

Waylite aggregate tends to produce a harsh-working concrete subject to serious bleeding. To overcome these difficulties, experiments were made with variations in grading, including gap gradings, variable additions of a fine sand, and additions of admixtures such as Cemixite and

TABLE 1—MECHANICAL ANALYSIS OF AGGREGATES

Sieve No.	Per Cent Retained on Given Sieve						
	Coarse Waylite		Janesville Gravel	Fine Waylite		Janesville Sand	University Sand
	Series A	Series B		Series A	Series B		
¾ in.	0	0	4.0	0	0	0	0
⅜ in.	11.3	1.4	61.6	0	0	0	0
4	95.0	77.4	94.4	0.5	0	1.3	0
8	97.7	87.2	100.0	16.8	19.5	10.2	0
16	98.1	92.8	100.0	38.5	47.5	25.5	1.0
30	98.3	95.6	100.0	63.2	74.5	48.0	4.0
50	98.5	97.0	100.0	82.4	88.0	85.0	40.2
100	98.8	98.2	100.0	92.3	94.0	98.8	89.7
Fineness Modulus	5.98	5.50	6.60	2.94	3.24	2.69	1.35

infusorial earth. Based on these experiments the following conclusions were drawn:

1—Scalping the fine Waylite, (removal of all material coarser than a No. 8 screen) markedly improved the workability and helped to reduce bleeding.

2—The addition of pulverized Waylite to regular or scalped fine aggregate did not improve workability enough to warrant its use economically.

3—By far the best results were obtained through the additions of a fine, river- or lake-type silicious sand to a properly designed mix. Satisfactory workability and almost complete bleeding control resulted when the sand addition was made equal to $\frac{1}{3}$ the total weight of the Waylite aggregate for the fluid mixes and $\frac{1}{5}$ the total weight for the vibrated mixes.

4—The judicious use of the two types of admixtures reduced the bleeding and helped in securing smooth action under the trowel but not so effectively as did the use of fine sand.

MIXING AND MANUFACTURE

All mixes were designed by the water-cement ratio trial method of proportioning. In these batches the aggregate and water were first thoroughly mixed before the cement was introduced and mixing was continued for several additional minutes after all materials had been added. This method must be followed if a satisfactory initial workability is to be obtained. The placeability of the Waylite mixes could not be measured satisfactorily by the slump test but was judged by the trowelling and rodding characteristics.

All aggregates were air-dried before using, and the water content of each mix reported was the total water added. This procedure was followed because of the difficulty of securing satisfactory absorption values for the cellular Waylite aggregate. Janesville and University sands absorb about 0.5 per cent water by weight, Janesville gravel about 1.4 per cent, fine Waylite and coarse Waylite, as nearly as could be determined by observation of the surface during slow drying after 30 minute immersion, 5.8 and 7.8 per cent respectively.

In Series B, the fluid Waylite mixes and the fluid sand-and-gravel mixes were designed to have approximately equal compressive strengths. The sand-and-gravel mixes had slumps varying between 4 and 5 inches. Each vibrated Waylite mix was designed to have the same total water as one of the fluid Waylite mixes. The vibrated mixes were satisfactorily placed with an internal vibrator operating at 10,000 r.p.m. and carried from 17 to 40 per cent more aggregate than did the fluid mixes.

All of the specimens were moist cured until tested.

TESTING

Standard methods were used for most of the tests and need no further comment. Since the details of the freezing and thawing tests are not standardized, the method used is briefly described:

After 28 days of moist curing, the 3 by 4 by 16-in. beams were placed in the dynamic testing apparatus and their fundamental frequencies of vibration obtained for two planes of vibration. Half of the specimens, controls, were then returned to the moist closet until the end of the test program, at which time the fundamental frequencies of vibration were again determined. They were then broken in cross-bending, and the fragments were tested as modified cubes in compression. The companion beams were soaked in water for 24 hours and were then placed in trays in which the water level was kept at ¾ in. at all times. The loaded trays were placed in a freezer in which the temperature varied between −5 F. and −12 F. and kept there until the beams reached a temperature of about −10 F. The trays were then taken out and the beams were air thawed at a temperature between 70 and 80 F. Such a cycle required 24 hours for completion. After every 10 cycles the beams were soaked for 24 hours in water after they had air thawed and the natural frequencies were again determined. Thus a history of the change of the natural frequency, or dynamic modulus of elasticity, was obtained for each beam, from which a prediction of the amount of damage caused by the freezing and thawing cycles could be obtained. When beams deteriorated to the point where they could be crushed between the fingers they were discarded. At the end of 50 cycles, the beams that were left were tested in the same way as the control specimens.

Dynamic moduli of elasticity were calculated by means of the equation*—

$$E = \frac{713.9\ p\ L^4\ f^2\ T}{m^4\ d^2}$$

where

p = specific weight in lb. per cu. ft.
L = length in inches
f = natural frequency in kilocycles
T = correction factor dependent on $\frac{d}{L}$ and Poisson's ratio
= 1.26 for d equal to 3 in., and 1.41 for d equal to 4 in.
m = constant = 4.73
d = depth of beam in plane of vibration, inches.

RESULTS OF TESTS—SERIES A

These results are summarized in Table 2. The data show that the compressive strengths, compressive moduli of elasticity, moduli of rupture, and specific weights decreased in the usual manner as the cement contents decreased. The specific weights of the green Waylite concrete varied from 106.2 to 112.5 lb. per cu. ft., with values for comparable mixes containing University sand additions in general slightly higher. Air

*Taken from paper "Discussion of Dynamic Methods of Testing Concrete with Suggestions for Standardization", by Leonard Obert and Wilbur I. Duvall, ASTM Proc., 1941.

TABLE 2—PROPERTIES OF LIGHTWEIGHT STRUCTURAL CONCRETE—SERIES A

Proportions by		Sacks of Cement per Cu. Yd. of Concrete	Total Water, Gal. per Sack of Cement	Specific Weight, Lb. per Cu. Ft.	Properties at 28 Days(*)				Remarks on	
Dry Weight Ct:F:C:USd	Dry Loose Vol. Ct:F:C:USd				Compressive Strength, psi	Modulus of Rupture, psi	Absorp., 24-hr. immersion, per cent	Compr. Modulus of Elasticity at Su/4, 10^6 psi	Workability	Bleeding
Aggregates—Fine and Coarse Waylite										
1:1.00:0.50	1:1.51:1.38	10.77	5.63	112.5	4940 (3890)	600 (825)	11.8	2.71	Good	Low
1:1.64:0.73	1:2.47:2.00	7.65	8.44	109.6	3200 (1760)	700 (435)	13.4	2.04	Fair	Low
1:2.25:0.75	1:3.40:2.06	6.18	11.26	107.7	1760 (920)	550 (280)	16.0	...	Poor	Medium
1:2.83:1.00	1:4.28:2.75	5.02	14.08	106.2	1470 (790)	340 (190)	16.9	...	None	High
Aggregates—Fine and Coarse Waylite plus 1/3 University Sand, by Weight										
1:0.67:0.55:0.41	1:1.01:1.51:0.38	10.41	5.63	113.2	4800 (3590)	810 (780)	10.9	2.70	Excellent	None
1:1.30:0.90:0.73	1:1.96:2.47:0.69	6.74	8.44	109.8	2950 (1810)	680 (470)	12.6	2.23	Very good	Very low
1:1.75:1.12:0.96	1:2.64:3.09:0.90	5.42	11.26	109.8	1690 (1000)	545 (255)	13.1	1.86	Good	Low
1:2.47:1.23:1.25	1:3.72:3.39:1.16	4.36	14.08	109.1	1120 (675)	310 (155)	14.0	....	Fair	Low
Aggregates—Fine and Coarse Waylite plus 1/5 University Sand, by Weight										
1:0.75:0.58:0.27	1:1.13:1.60:0.25	10.30	5.63	111.2	4580 (3300)	890 (675)	11.4	....	Excellent	None
1:1.46:0.84:0.46	1:2.20:2.31:0.43	6.99	8.44	109.7	2160 (1310)	595 (430)	13.8	....	Good	Very low
1:1.98:1.12:0.65	1:2.98:3.09:0.61	5.39	11.26	107.7	1740 (970)	445 (290)	15.0	....	Fair	Low
1:2.80:1.17:0.79	1:4.22:3.21:0.75	4.40	14.08	107.3	1200 (710)	305 (160)	14.9	....	Fair	Medium

*Values in brackets are for 7-day results.

drying reduced these values to between 90 and 100 lb. per cu. ft. Compressive moduli of elasticity, varying from 1,860,000 to 2,710,000 psi, are approximately two-thirds as large as would be expected for gravel concretes of similar strengths. The ratios of modulus of rupture to

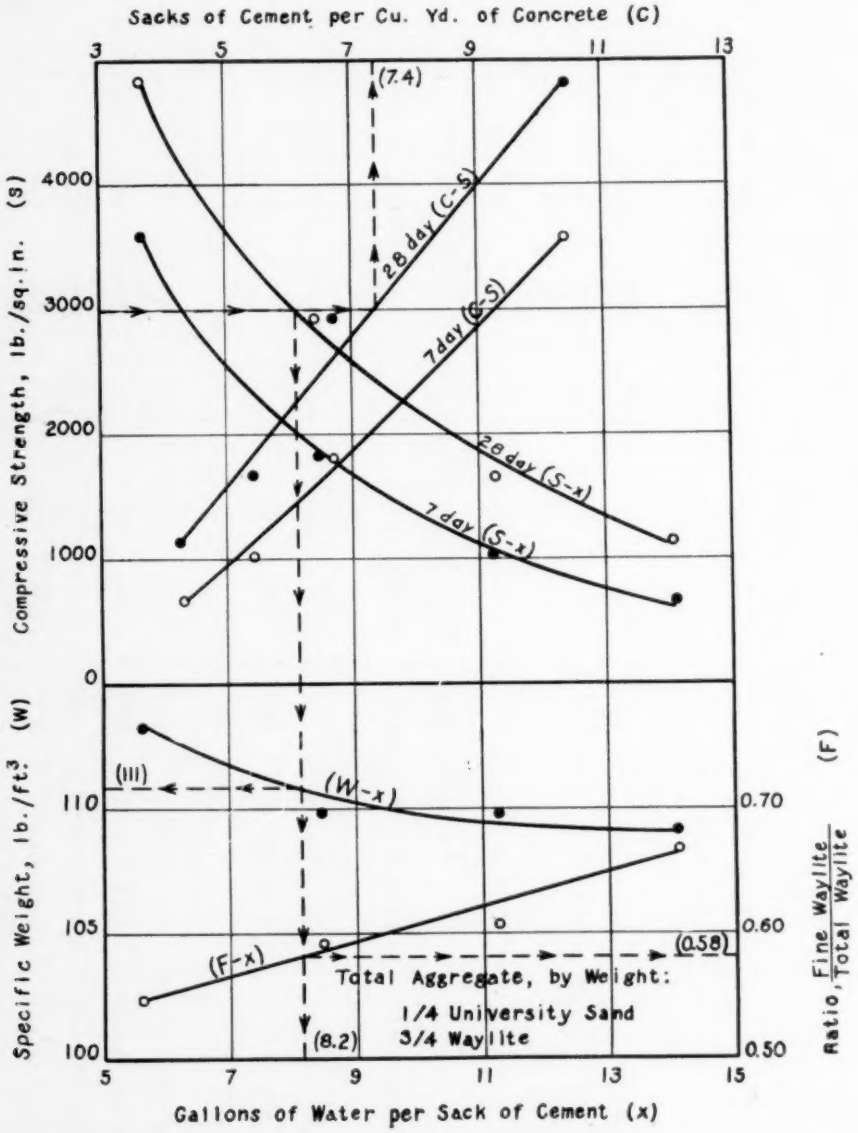

Fig. 1—Compressive strength design curves for fluid Waylite concrete with 1/3 University sand added

compressive strength and the strength gains from 7 to 28 days closely approximate those generally reported for gravel concrete. The 24-hour absorptions of the plain Waylite concrete varied from 11.8 to 16.9 per cent and were generally slightly higher than for the Waylite concrete with University sand additions. These absorpitons are about twice as high as those expected for gravel concrete of comparable strengths.

Commonly used water-cement ratio design methods may be used successfully with this aggregate. Typical results are shown for the studies of this series in Fig. 1. From such curves suitable mixes for any strength specification can be computed quickly, as shown by the dotted lines for a mix with a 3000 psi compressive strength at 28 days. These curves call for 7.4 sacks of cement per cubic yard of concrete, 8.2 gal. of water per sack of cement, and a ratio of fine to total Waylite of 0.58. From the specific weight and other known quantities, the batch weights are easily calculated.

RESULTS OF TESTS—SERIES B

Strength, toughness, and durability characteristics for fluid and vibrated Waylite concrete and fluid gravel concrete are presented in Tables 3 and 4. The strength values in this series are generally lower than those of Series A, largely because a somewhat coarser and lower strength cement was used. On an equal total water basis, the vibrated Waylite concrete carried from 17 to 40 per cent more aggregate than the fluid Waylite concrete, and its specific weight was from 2.4 to 5.6 lb. per cu. ft. less because it carried less University sand and less cement per unit volume. The green Waylite concrete weighed about 75 per cent as much as did the green gravel concrete, while on a dry basis the ratio would be under 70 per cent. Properties of the vibrated Waylite and fluid Waylite concretes, for a given total water content, were essentially similar. The fluid Waylite concrete required about 1.5 times as much cement for a given strength, as did the fluid gravel concrete. Modulus of elasticity values for the gravel concrete were about 45 per cent higher than those for Waylite concrete of similar strength. Information on the effect of span length on modulus of rupture indicates that the value determined from a test on a 14-in. span is approximately 1.2 times that determined on a 36-in. span for the Waylite beams, and about 1.4 times as high for the gravel concrete beams. Strength gains for moist curing from 28 to 90 days, both in compression and transverse bending (Table 4) are similar for both types of concrete, with slightly larger percentage increases obtaining for the Waylite concrete and particularly for the leaner mixes.

TABLE 3—PROPERTIES OF FLUID AND VIBRATED LIGHTWEIGHT STRUCTURAL CONCRETE AND FLUID SAND-AND-GRAVEL CONCRETE AT 28 DAYS—SERIES B

Proportions by		Sacks of Cement per Cu. Yd. of Concrete	Total Water, Gal. per Sack of Cement	Specific Weight, Lb. per Cu. Ft.	Compressive Strength, psi	Properties from Transverse Tests on 3 by 4 by 39-in. Beams			
Dry Weight Ct:F:C:USd	Dry Loose Vol. Ct:F:C:USd					Energy of Rupture*, in lb. per cu. in.	Modulus Elasticity at Su/4, 10^6 psi	Modulus of Rupture, psi	
								36-in. span	14-in. span
Fluid Waylite Concrete with 1/3 University Sand									
1:0.59:0.49:0.36	1:0.99:1.16:0.35	11.55	5.62	118.3	4090	0.0099	2.86	655	840
1:0.80:0.63:0.48	1:1.34:1.49:0.47	9.62	6.53	116.8	2880	0.0085	2.53	585	650
1:1.10:0.80:0.63	1:1.84:1.89:0.61	7.50	8.10	110.8	2200	0.0054	2.51	435	525
1:1.63:1.00:0.88	1:2.73:2.36:0.86	5.69	10.57	107.9	1260	0.0031	2.36	275	345
Fluid Janesville Sand-and-Gravel Concrete									
1:1.70:2.38	1:1.53:2.47	7.69	5.85	150.0	4590	0.0116	4.02	680	890
1:2.57:3.34	1:2.31:3.47	5.68	6.98	148.9	2660	0.0067	3.84	480	660
1:3.23:3.98	1:2.90:4.13	4.80	8.33	149.5	2050	0.0044	3.80	390	535
1:4.04:4.88	1:3.64:5.07	3.95	10.35	148.9	1400	0.0031	3.27	265	400
Vibrated Waylite Concrete with 1/5 University Sand									
1:0.94:0.74:0.34	1:1.57:1.75:0.33	9.20	5.62	112.7	3850	0.0120	3.20	715	850
1:1.23:0.92:0.43	1:2.06:2.17:0.42	7.64	6.53	110.6	2370	0.0088	2.62	585	700
1:1.52:1.05:0.52	1:2.54:2.48:0.51	6.40	8.10	107.2	1640	0.0066	2.49	460	525
1:2.15:1.27:0.68	1:3.60:3.00:0.66	5.02	10.57	105.5	1260	0.0034	1.91	295	370

*Calculated energy of rupture = $\frac{\text{Area under load-deflection curve}}{\text{Volume of specimen between supports}}$.

TABLE 4—EFFECTS OF FREEZING AND THAWING ON THE PROPERTIES OF FLUID STRUCTURAL CONCRETE—SERIES B

Proportions by Dry Weight Ct:F:C:USd	Sacks of Cement per Cu. Yd. of Concrete	Properties at 28 days(1)			Increase in Dynamic E for Moist Cured Control Beams from 1 to 3 mos., per cent	Per Cent Decrease in Dynamic E After Freezing and Thawing Cycle No.					Decrease in Modulus of Rupture after 50 Cycles, per cent(2)	Decrease in Compressive Strength after 50 Cycles, per cent
		Compressive Strength, psi	Modulus of Rupture, 14-in. Span, psi	Dynamic Modulus of Elasticity, 10^6 psi		10	20	30	40	50		
Waylite Concrete with 1/3 University Sand												
1:0.59:0.49:0.36	11.55	4090 (4910)	840 (930)	3.59	1.2	2.3	11.3	39.3	82.4	90.4	89.9 (89.5)	55.6
1:0.80:0.63:0.48	9.62	2880 (4500)	650 (845)	3.30	4.4	10.6	56.0	84.8	93.1	....	....	....
1:1.10:0.80:0.63	7.50	2200 (3500)	525 (685)	2.81	5.7	31.9	81.6	94.5	....	....	....	....
1:1.63:1.00:0.88	5.69	1260 (1990)	345 (495)	2.23	13.4	73.1	90.2	....	....	....	....	....
Sand-and-Gravel Concrete												
1:1.70:2.38	7.69	4590 (5730)	890 (1005)	5.87	4.1	10.8	18.9	24.3	39.1	43.5	64.9 (57.7)	22.2
1:2.57:3.34	5.68	2660 (4330)	660 (830)	5.49	6.0	29.6	49.4	63.9	82.2	90.2	88.5 (89.4)	36.9
1:3.23:3.98	4.80	2050 (3340)	535 (660)	5.19	11.4	41.4	69.4	87.6	....	....	....	....
1:4.04:4.88	3.95	1400 (2230)	400 (435)	4.45	14.5	66.4	91.2	....	....	....	....	....

(1) All specimens moist cured. Values in brackets are 90-day results.

(2) Bracketed values are predicted decreases in moduli of rupture calculated from $R = 6\ E^{0.6}$, (Reagel's formula) where R = percentage reduction in modulus of rupture, E = percentage reduction in dynamic modulus of elasticity.

Comparison of the dynamic moduli of elasticity with the cross-bending moduli shows that for values of E around 2,000,000 psi, good agreement was obtained, but the dynamic modulus was about 50 per cent higher than the cross-bending E for values of the latter around 4,000,000 psi. Moist curing of the control beams from 1 to 3 months increased dynamic E from 1.2 to 13.4 per cent for the Waylite concrete, and from 4.1 to 14.5 per cent for the gravel concrete. The greatest increases were obtained in each case for the weakest concrete. Damage due to freezing and thawing, indicated by decreases in dynamic E, is shown for each 10 cycles. The Waylite concrete beams generally resisted the early cycles better than did the gravel concrete beams, but thereafter lost their structural strength much more rapidly. After 50 cycles, only the strongest set of Waylite beams was left, and had suffered a 90.4 per cent decrease in dynamic E, while beams representing the two richest mixes for the gravel concrete still remained. These had suffered decreases in dynamic E of 43.5 and 90.2 per cent. Beams left after 50 cycles were tested and their compressive strengths and moduli of rupture obtained. Observed decreases in moduli of rupture agreed closely with values predicted by Reagel's formula from decreases in the dynamic moduli of elasticity (Table 4).

The method of disintegration of the Waylite concrete beams was different from that of the gravel concrete beams. The first evidence of disintegration of the gravel concrete beams was crazing of the surface elements, followed by softening of the ends. The softening progressed toward the center until the beams were structurally worthless. Crazing was also evident in the first stages of disintegration of the Waylite concrete beams, followed by softening of the ends along with a softening action near the centers of the sides. This was followed by cracking which seemed to emanate from the softened areas, and by cracks that started from the top surfaces and progressed downward. Frequently final failure was accompanied by a wide crack that started from the bottom and extended almost to the top. Fig. 2 shows Waylite beam A5 after 50 cycles of freezing and thawing.

RESULTS OF REINFORCED CONCRETE BEAM TESTS

Properties of eight reinforced Waylite and gravel concrete beams and their controls are shown in Table 5. Noteworthy is the close agreement, at $0.45\,f_c'$, between stresses determined from measured strains and those calculated from standard theory assuming straight line variation of the concrete stress. This agreement persisted to ultimate for both concrete and steel, concrete stresses for measured strains being evaluated from compressive stress-strain diagrams for cylinders. The ultimate tensile

Fig. 2—Appearance of a rich Waylite beam after 50 cycles of freezing and thawing

strength of the Waylite concrete in beams 1 and 2, based on the calculated shear stresses at ultimate, assuming the concrete fully effective in tension near the end of the beams, was approximately 5 per cent of the compressive strength. This low value was also checked by a limited series of tension tests on 6 in. cylinders where the tensile strengths developed varied from 4½ to 6 per cent of the compressive values. With due allowance for variations in compressive strength, bond resistance was similar for both types of concrete, the values reported agreeing closely with those reported by others for substantially similar bars and concrete strengths.

It would appear that reinforced lightweight concrete may be designed safely according to the principles now accepted for heavy concrete except that more shear steel must be used because of the lower tensile-compressive strength ratio for lightweight concrete. Attention should be called to the markedly lower values of modulus of elasticity for lightweight concrete which means an increased value of k and hence a greater percentage of tensile steel to develop a given concrete strength as compared with gravel concrete.

CONCLUSIONS

1—Water-cement ratio design methods may be applied with confidence to the design of lightweight structural concrete mixes for both puddled and vibrator placement provided special precautions are observed (a) to mix aggregate and water *before* cement is added to the mix, (b) to control water in the aggregate with accuracy and (c) to provide thorough mixing.

2—Satisfactory workability and bleeding control may be secured effectively through additions of a fine silicious sand to a properly designed mix.

TABLE 5—TESTS OF REINFORCED CONCRETE BEAMS

TEST RESULTS FOR 6 in. by 12 in. by 9 ft. 6 in. BEAMS ON 9 ft. SPAN

Beam No.	Longitudinal Steel, per cent	Spacing of ¼ in. Stirrups, in.	Calculated Stresses at 0.45 f_c', psi						$\frac{M}{bd^2}$ at Ultimate	Type of Failure
			Compression		Tension		Shear	Tension in Stirrups, from Strain		
			Theory	Strain	Theory	Strain				
1	2.11	None	1430	1500	17800	16300	110	....	370	Shear
2 Waylite	2.11	None	1680	1880	20300	18400	126	....	445	Shear
31 plus 1/3	2.11	5	2070	2170	24700	24000	152	2000	825	Tension
32 University	2.11	5	1890	2150	22400	24000	135	8000	810	Tension
41 Sand	1.68	5	1280	1280	19500	17900	96	0	650	Tension
42	1.68	5	1240	1240	18000	14600	92	0	645	Bond
51 Janesville	1.92	4	2740	2910	26000	22800	156	0	870	Tension
52 Sand & Gravel	1.89	4	2320	2560	24100	22800	138	0	815	Tension

(B) PROPERTIES OF CONCRETE AND STEEL

Beam No.	Compressive Strength, f_c', psi	Modulus of Elasticity at 0.45 f_c', 10^6 psi	Calculated Bond Stresses, psi					No. and Diameter Deformed Bars	Yield Point of Steel, psi
			Pullout from 6 in. by 6 in. cyl.			Beam Test			
			Slip 0.0001 in.	Slip 0.001 in.	Ultimate	Slip 0.0001 in.	Slip 0.001 in.		
1	3180	2.20	305	830	1200	60	...	3—¾ in.	46,500
2 Waylite	3740	2.40	280	835	1095	65	...	do	for ¾ in.
31 plus 1/3	4600	2.44	285	970	1160	55	180	do	bars;
32 University	4210	2.43	265	860	995	65	175	do	49,100 for
41 Sand	2850	1.98	265	565	870	100	200	2—⅝ in.	⅝ bars;
42	2790	2.09	265	670	840	80	180	1—¾ in.	50,470 for
51 Janesville	6100	4.65	450	1335	1750	140	...	2—¾ in.	¼ in.
52 Sand & Gravel	4730	4.28	690	1165	1630	140	...	1—⅝ in.	stirrups

3—The specific weight of Waylite concrete is about 70 per cent, the absorption about twice, and the modulus of elasticity about two-thirds that of gravel concrete of similar strength in the range up to 4500 psi.

4—As the compressive strengths increase up to 4500 psi the bond strengths also increase, closely checking values for gravel concretes of similar strength.

5—The toughnesses of fluid Waylite and gravel concretes of similar strengths measured in transverse bending are substantially equal, ranging from 0.003 to 0.010 in. lb. per cu. in. for compressive strengths from 1250 to 4500 psi.

6—Vibration with properly designed mixes is highly effective in increasing strength, toughness and stiffness of lightweight concrete for a given cement content.

7—Fluid Waylite concrete requires about 50 per cent greater cement content than gravel concrete of similar placeability to produce a given strength.

8—The change in dynamic modulus of elasticity is a rapid and reliable means for measuring damage caused by freezing and thawing.

9—Structural Waylite concrete offers somewhat greater resistance to a limited number of freezing and thawing cycles than does gravel concrete but breaks down much more rapidly as the number of cycles becomes large. Except for very rich mixes, structural Waylite concrete should not be recommended for placement in locations where considerable freezing and thawing action in the presence of moisture may take place unless an excellent waterproofing treatment is provided.

10—A 3 by 4 by 16-in. beam proved to be a highly satisfactory specimen, permitting dynamic E determinations in two planes, one serving as a check on the other. Transverse and compressive strengths could also be secured from the same specimen.

11—Reinforced Waylite concrete may be designed according to the principles now accepted for heavy concrete except that more shear steel must be used because of the somewhat lower tensile-compressive strength ratio for lightweight concrete, and due allowance must be made for the lower values of modulus of elasticity.

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Discussion of a paper by George W. Washa and Kurt F. Wendt:

The Properties of Lightweight Structural Concrete Made with Waylite Aggregate*

DISCUSSION By JOHN S. NELLES and the AUTHORS

By JOHN S. NELLES†

A study of this excellent paper verifies the fact that lightweight structural concretes are costly in cement when compressive strengths of 2000 psi. or greater are required.

Any concrete mixture containing a light, porous aggregate requires more cement-water paste to fill the voids in the aggregate and between the aggregate particles, than do the standard concretes. If the quality of the paste is to be maintained it is very costly in cement, unless another material can be found of similar physical characteristics to replace part of the cement. If the paste is diluted by the addition of water only, the water cement ratio will be increased, the strength of the concrete decreased, and the workability and density ruined because the paste will not have sufficient viscosity to keep the aggregate particles in suspension.

When fine aggregate of sand size, but of greater specific gravity than the coarse aggregate is added, the heavier fine aggregate will settle out of the mix and float the lighter coarse aggregate during the placing of the concrete; and also during the sedimentation cycle after placement.

Using a fine aggregate, obtained by crushing the light coarse aggregate to a sand size would make a more homogeneous mix, because the fine and coarse aggregates are of the same type and specific gravity; but would not improve the workability or water gain characteristics to any great extent because the individual particles of the manufactured fine aggregate would be angular in form, similar to the harsh coarse aggregate.

Pulverizing this aggregate and adding the powdered material would result in only a slight improvement, because the angularity of the particles would permit only a limited amount of this material to be added, whereas

*ACI JOURNAL, June 1942; *Proceedings*, V. 38, p. 505.
†Detroit Edison Co., Detroit, Mich.

an excess of fine material is needed to augment the paste content. This would explain why the addition of pulverized "Waylite" to the mix did not give entirely satisfactory workability.

What light-weight concretes require is a fine aggregate that will produce an excess of paste, rather than an excess of mortar, and more particularly, a paste that will have sufficient viscosity to hold the coarse aggregates in suspension.

For the past twelve years we have been investigating the value of fly-ash in concretes and much of that work had to do with fly-ash in cinder concrete.

Fly-ash is an extremely fine material, (73 per cent passing a 400-mesh Tyler screen), and weighs 53 lb. per cu. ft. It is collected from the stacks of pulverized coal-fired boilers and its particles are spherical in form. This unusual physical characteristic makes it possible to add very large amounts to the mix, increasing the amount of paste, enabling the harsh, coarse aggregate particles to move more freely in the mass without interference, thereby greatly improving the workability. At the same time the fly-ash increases the viscosity of the paste, just as the addition of more cement would do. For this reason, fly-ash, when added in sufficient quantity, not only serves as an excellent fine aggregate, but also because of its paste-forming characteristic, makes it unnecessary to add such abnormal quantities of cement.

It would be out of place to publish any comprehensive detail of our tests in this discussion, but since we have actually used several thousand cubic yards of cinder-fly-ash concrete, we include data on a few of these mixes.

TABLE A—CINDER FLY-ASH CONCRETE

Nominal Mix

Cement Content Bags/Cu. Yd.	Fly-Ash Content Lb./Cu. Yd.	Cement Bags	Fly-Ash Cu. Ft.	Crushed Cinders Cu. Ft.	Water (Gal. per Bag)	Compressive Strength 28 Days (psi)
5.02	467	1	1.86	4.66	11.9	3075
4.13	512	1	2.48	6.33	12.9	2120
5.05	505	1	2.0	4.16	13.1	3130
5.50	500	1	1.82	3.92	11.5	3270

When light-weight structural concretes are produced which will compare favorably with standard concretes, as to compressive strength, workability and cement content, their advantages in light-weight and increased elasticity will bring them into more extensive use in the build-

ing industry. We believe that the use of fly-ash as a fine aggregate will be a major factor in bringing this about.

AUTHORS' CLOSURE

The authors are greatly interested in the discussion by Mr. Nelles regarding the use of fly-ash in lightweight concrete. At the present time we are both actively engaged in some work involving additions of fly-ash and also extremely finely divided Waylite to structural Waylite concrete. Unfortunately our work has not progressed far enough to allow us to present any definite conclusions.

With regard to the statement by Mr. Nelles that: "When fine aggregate of sand size, but of greater specific gravity than the coarse aggregate is added, the heavier fine aggregate will settle out of the mix and float the lighter coarse aggregate during the placing of the concrete; and also during the sedimentation cycle after placement," it would appear from our experience that the mixes to which he refers have been improperly designed. A well designed lightweight concrete with a fine heavy sand added for workability, in which the water is closely controlled and which has been properly and sufficiently mixed, is plastic, works easily, and shows little or no segregation or excess water after placement. Certainly no such flotation as described has ever been observed by us. Thousands of yards of structural Waylite concrete using fine silicious sand additions have been placed by hand puddling and by vibratory methods during the past year with satisfaction.

In his last paragraph, Mr. Nelles refers to the advantages of lightweight concrete and among them he names increased elasticity. It is not clear just what is meant by the term in this instance, certainly not modulus of elasticity.

We appreciate that the addition of fly-ash to light weight concrete made with angular aggregates will undoubtedly alleviate some of its undesirable features. That it will do so more effectively and economically than fine silicious sand or other admixtures has not yet been demonstrated to the authors' satisfaction. It is hoped that the projected investigations will result in positive conclusions on this point.

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Job Problems and Practice

WHAT JPP MEANS

—an informal clearing house for mutual help on concrete problems; an opportunity for the ACI member who *knows* to tell the ACI member who *needs* the information. If the subject matter holds probabilities of general interest it will be briefed in these pages—with authorship credit unless the contributor prefers not.

If you know of an interesting problem whose solution has smoothed the way of someone in the field, record it briefly. If you have a problem, present it; a question, ask it. If you are moved to helpful comment or constructive criticism write the Secretary.

The "answers" are those of individuals to whom the questions are referred and not of the ACI.

Concrete Curing Compound—Made on the Job (38-100)

BY PAUL A. JONES*

After a good deal of experimenting, we of the Buffalo Rapids project, Bureau of Reclamation, have hit upon what we consider to be a highly satisfactory concrete curing compound that can be cheaply made on the job. Realizing that the popularity of liquid sealing compounds for curing concrete is increasing rapidly, and that with present raw material shortages the supply of reliable commercial products might become limited to the point of restriction, we offer our experiences with P. J. emulsion concrete curing compound.

First, a few words on the development of P. J. emulsion. With a rather vague idea of the composition of one type of commercial compound as a background, it was our first thought that a material saving in cost might be achieved by dispersing the solids portion of a curing compound in water as an emulsion, rather than to use a solvent. Triethanolamine in combination with stearic acid was selected as the emulsifying agent. Next, having determined on paraffin as an inexpensive major ingredient of the solids portion, we devoted considerable time to tests, with the

*Construction Engineer, Bureau of Reclamation, Buffalo Rapids Project, Montana.

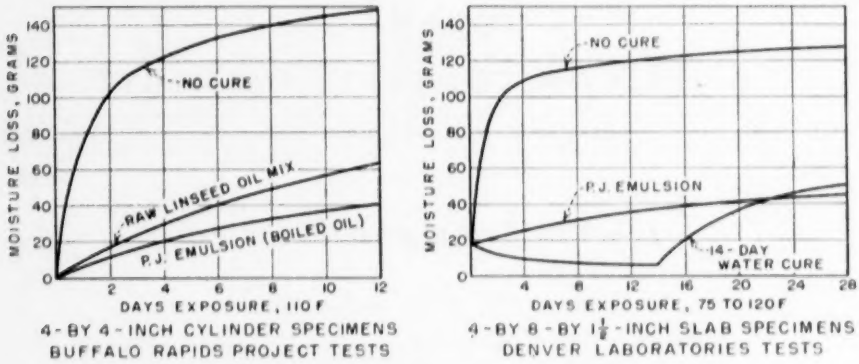

Fig. 1—Effectiveness of P. J. emulsion as a moisture seal

object of finding a material for combination with paraffin that would impart elasticity as well as tightness to the film. Glue, varnish, motor oil, neatsfoot oil, and linseed oil, to mention a few, were tried. It became evident before long that boiled linseed oil showed the most promise, so we concentrated on this and finally concocted a mixture that provided a high degree of moisture-sealing efficiency, coupled with low material cost.

After thorough tests indicated that something really worthwhile had been discovered, we turned our findings over to the main Bureau of Reclamation laboratories in Denver for verification. The Denver results checked with ours and approval was granted for the use of P. J.† emulsion on our concrete work.

As it is not desired to encumber this article with detailed test data, there are presented just two typical graphs for the edification of those who might want to visualize the moisture-sealing efficiency of the emulsion. Note that boiled linseed oil is considerably more effective than raw oil, and is well worth the extra few cents per gallon.

Now for the secret formula:

To make 50 gallons

Paraffin	49.7 lb.
Boiled linseed oil	74.5 lb.
Triethanolamine	13.2 lb.
Stearic acid	16.6 lb.
Water	248.3 lb.
Total	402.3 lb.

(Boiled linseed oil from any reliable dealer may be used. Stearic acid may be of the single-pressed grade. Paraffin of 125-127 m.p., refined grade is suitable. It is understood that triethanolamine is still available in small quantities in chemical supply houses, but the manufacturers have recently been obliged to require a preference rating in order to furnish this material.)

†Dubbed "P. J." for Paul Jones by the Denver office.

The preparation of P. J. Emulsion is simple. All ingredients except triethanolamine are heated together until the solid constituents are melted, the mixture being slowly stirred during this operation. The triethanolamine is then added and the stirring continued until the mixture cools. The stirring may be done by hand, using a paddle, or, as on our work, may be accomplished by an electrical mixer designed for slow rotation—about 18 r.p.m. Too rapid stirring will result in an unstable emulsion. Usually, any slight separation can be rectified by reheating and stirring.

The formula as given is particularly suitable when soft water is used as an ingredient. If hard water only is available, some adjustment in the total or relative quantities of triethanolamine and stearic acid will be necessary to secure stability. In most cases the necessary alteration can be determined by trial and error, using small test batches. It is desirable to store P. J. emulsion at a temperature of about 60°F., but, if it should freeze, there is evidence that no permanent harm has been done, and that reheating and mixing will restore the original properties.

Application to an ultimate coverage of 150 square feet per gallon is recommended for most concrete curing, and may be accomplished by either spraying or brushing. The emulsion will produce some discoloration on concrete surfaces, and might be objectionable for that reason for some structures. If efficient heat reflection for structures exposed to direct sunlight is desired, it is suggested that the emulsion film be coated with whitewash, although we have not had occasion to use this procedure on our work as yet.

As a last thought, it might be stated that P. J. emulsion has proved to be an effective substitute for form oil. We have been using it for some time for that purpose on both wood and steel forms.

Concrete of the Future (38-101)

BY R. E. DAVIS*

If we review the history of concrete I think we will all appreciate that there has been a gradual progress, not always a smooth progress, but a progress in jumps. I do not believe we have yet come to the point where we are fully realizing the potentiality of concrete as a construction material. I think we are a long way short of it. Take strength of concrete, for example. I remember when we used to talk about twelve hundred pound concrete. Now we talk about five thousand pound concrete, and we know that we are obtaining much higher strengths than that, even, in concrete cast in place. I have in mind a test which I witnessed something over a year ago in the laboratory of the Bureau of Reclamation

*In response to a question, at the 1942 ACI "Quiz Session," as to the "potentialities of concrete."

in Denver—a core from a dam containing a barrel of cement per cubic yard exhibited a compressive strength of a little better than 8000 psi. —that is something for us to think about. Many of our building stones exhibit strengths of 15,000 or 20,000 psi, a few of them as high as 25,000 lb. I see no reason to doubt that we may have concretes which will quite commonly go to 10,000 psi, perhaps some of them to 15,000 or 20,000—probably not with our present cements but perhaps with our present cements modified by added materials.

When we begin to think about the other properties of concrete, we all realize that we need a concrete that is more rubbery. In a meeting yesterday somebody spoke of the desirability of having cement paste that is not so brittle so that we might get rid of shrinkage cracks, one of the most troublesome things with which we have to contend. I believe we may reasonably expect that in time there will be developed a concrete which will produce buildings and bridges and dams free from cracks. We are still looking for a concrete that will resist the action of weather as well as do some of our natural stones.

The optimums in high strength, volume constancy, freedom from cracking, weathering resistance and other desirable properties are not going to be achieved today or tomorrow but so long as we are not satisfied with our achievements, so long as the men in the laboratory and field are searching for improvements, development will continue. One of the most interesting things about this material of concrete is that there is never any lack of problems, we have never come to the point where we can say our work is finished; we always have something to look forward to.

I think it is not unreasonable to expect that, both in the quality of the material and in the variety of uses to which it is adapted, the concrete of the future will be as superior to that of today as the concrete of today is superior to that of the turn of the century. In other words, we may expect future developments to be just as great as have been those of the past.

Placing Cold Concrete in Hot Weather* (38-102)

ROY W. CARLSON†

Q—(a) What are the advantages of placing cold concrete in hot water? (b) How, with efficiency, may concrete be made and placed cool in hot weather?

A—The advantages of placing concrete at low temperature in hot weather are that the maximum temperature is thereby reduced and the chance of getting cracks due to subsequent cooling are eliminated or

*From the 1942 ACI Convention's "Quiz Session."
†Mass. Institute of Technology, Cambridge, Mass.

at least modified. I will not go into all the ramifications of how and why this is so, but I think most of you know that in mass concrete, especially in dams, it is important to keep the maximum temperature down. The only really effective way to place cold concrete seems to be to add slush ice to the mixer as part of the mixing water. This is being done in a number of dams, for example in the Friant Dam in California and the Norfolk Dam in Arkansas. This subject is treated at length in the report of ACI Committee 207, Properties of Mass Concrete,, soon to be released.

Evaluating "n" (38-103)

Charles A. Ellis* rose at the ACI 1942 "Quiz Session" with a question from the floor; Arthur N. Talbot presented the answer. The question:

Q—Some years ago this Institute sponsored a new formula for reinforced-concrete columns in which n seems to have been thrown in the ashcan. The formula tacitly admits that the moduli of elasticity of steel and concrete are the same, or else that there is no relation whatever between unit stress and unit strain. If there is no relation between unit stress and unit strain, I shall probably lose my job because I am teaching the theory of statically indeterminate structures, which theory is developed upon a relation between unit stress and unit strain.

I now explain the new column formula to the students by what I call the Alphonse-Gaston method. When the concrete gets tired, it throws the load into the steel, and when the steel gets tired it throws the load back into the concrete, and I get away with it. Now comes another perplexing question in Section 804 (c) of the Joint Committee Report, which states:—"Where compression reinforcement is used its effectiveness in resisting bending may be taken at twice the value indicated by the computations . . . ." The student asks "Professor, why twice?" What shall I tell him? Why twice? Is the question clear?

BY A. N. TALBOT†

A—I do not know the origin of this provision; until about an hour ago I had not known that it was in the design section of the ACI Building Regulations. In recent years I have been doing and thinking about other things than reinforced concrete, but while Professor Ellis was speaking, I recalled some things that happened in 1907,‡ in connection with tests of plain and reinforced concrete columns, and in connection with some work that had been done in France. I myself devised a little

*Dept. of Civil Engineering, Purdue University, Lafayette, Ind.

†Doctor Talbot's last participation in Institute discussion—he died April 3. See *News Letter* pages in this and April JOURNALS.

‡See "Tests of Concrete and Reinforced Concrete Columns," by Arthur N. Talbot, Bulletin 10, Engineering Experiment Station, University of Illinois, 1907.

analytical consideration of the compressive strength of concrete in flexure, involving the relations between stress and strain, and between steel and concrete; the result was called the parabolic formula. Now in the theory of the parabolic formula, the initial tangent to the parabolic curve (representing the relation between concrete stress and strain) is the line of constant or initial modulus of elasticity. If the stress in the concrete is small, the parabola remains close to the tangent, and the concrete stress closely approaches that calculated from the initial modulus of elasticity. Now if the load is increased so that the concrete reaches its full resistance, the stress is indicated by the maximum ordinate of the parabola, and at that point the stress is just half as much as might be calculated from the strain and the initial modulus of elasticity. In other words, at the point of failure of the concrete, if the concrete follows the parabolic theory of stress distribution, the effective (or secant) modulus of elasticity of the concrete is just one-half the initial modulus; the corresponding value of n at this point of failure is twice the initial, or elastic, value. So it may be that this code provision originated in a study of stress-strain relations at failure according to the parabolic theory. Surely it would be on the safe side if the material were to act in such a way as to follow such a particular curve. Whether it is right or not, I don't know.

BY F. E. RICHART*

I believe Professor Talbot has indicated the essential consideration that led to the code provision under discussion, viz., the plastic or inelastic deformation of the concrete in compression and flexural members at failure. Many tests have shown that compressive reinforcement may develop stresses under steady load that are three to four times the initial elastic stress; furthermore, when the concrete is highly stressed it exhibits a short-time creep which completely upsets the conventional computation of compressive steel stress based on the initial modular ratio, n. In Europe, it is quite common to use a rather large value of n, of say 15 to 20, although the value based on elastic action might be only 8 to 10. Since the stress in compression steel is a direct function of n, it is evident that while no theory is particularly accurate, the use of an increased value of n will come nearer expressing actual conditions at failure than will the conventional value. Other properties of the beam or column, such as the position of the neutral axis, compressive stress in concrete, etc., are less directly affected by n, and much confusion is avoided by making no change in the use of the conventional n in these cases.

The provision for using a double value of n in computing stress in compressive steel in beams while retaining the ordinary n for other

*Subsequently, by letter.

purposes is doubtless repugnant to anyone who thinks the conventional straight-line theory represents the actual stress distribution in a beam, particularly near the ultimate load. This provision is intended to bring design and actual stresses into closer agreement than before. The general subject of time yield and inelastic deformation of concrete and of the consequent high compression steel stress, and the recent discussions both in Europe and America of various plastic theories of beams have drawn the attention of designers and investigators to the need for a more realistic treatment of beam design; however, I agree with Professor Ellis that the inclusion of a new code provision such as this one needs explanation and justification. The foregoing represents my idea of the reasons for this provision of the code.

Determining Reinforcement in Hardened Concrete (38-104)

Q—Our problem is in the fact that we propose to install heavy equipment in an old building with reinforced concrete floors and beams. Apparently no drawings are available that show the reinforcing details. It becomes highly important that we find some means of determining the amount and distribution of the reinforcing steel. We have been advised that an X-ray process has been successfully developed for this purpose. We should greatly appreciate any information or suggestions you may have to offer.

BY H. L. FLODIN*

A—Some work of this type has been done both in this country and in Europe.

At the annual meeting of the American Society for Testing Materials, held in 1936, a symposium on this subject was presented. Not published in the regular proceedings of the Society this was published in a separate book under the title "Symposium on Radiography and X-ray Diffraction Methods." On page 96 there is an X-ray photograph of a reinforced concrete floor 5 in. thick which not only shows the embedded bars very clearly but even the fine wire ties at the intersections of the bars. There is a brief description of the method used on page 97. The information was supplied by Herman E. Seemann, physicist of the Kodak Research Laboratories, Rochester, New York.

In a discussion of Mr. Seemann's paper by R. K. Bernard, consulting engineer of Philadelphia, which appears on page 112, Mr. Bernard describes X-ray studies to determine the causes of cracks in certain reinforced concrete roof girders. This discussion is illustrated with complete diagrams of the structure in question, with sketches showing the positions of the X-ray lamp. The photographs obtained are reproduced

*Structural Bureau, Portland Cement Assn., Chicago.

and demonstrate very clearly that certain reinforcing bars supposed to be placed in these girders, and particularly the stirrups, were wholly missing. Thus, the crack was completely accounted for. There is also a photograph showing the setup of the X-ray apparatus on the job. This appears to be reasonably simple.

Some time ago we discussed the subject with representatives of the General Electric X-Ray Corp., Chicago, who stated that the method was entirely feasible and so far as they knew would be successful in photographing thicknesses of concrete at least up to 12 in. Beyond this point they were inclined to think that the strength of current required would make the apparatus too cumbersome for field use. We were referred to the National Testing Laboratories, Rochester, New York. These laboratories were prepared to undertake any reasonable field work and have had considerable experience in diverse industrial applications.

Several references have appeared on the subject in foreign publications. One of these is, "X-Ray Studies of Concrete" by C. Kantner in *Beton und Eisen*, Vol. 31, page 170, for June 5, 1932. Five photographs of a concrete beam are shown to indicate the effectiveness of this method of investigation for locating reinforcing.

Another article appears under the title "Investigations of Reinforced Concrete with X-Rays," by R. Bernhard in *Zentralblatt der Bauverwaltung*, Vol. 51, page 806, for Dec. 30, 1931.

Instead of trying to locate the reinforcing and the amount of steel, load tests are often made to determine the carrying capacity of floor slabs. These tests are made by loading the floors with pig iron or bagged sand or other heavy materials and the deflection is measured. The work, of course, should be done by competent structural engineers.

A Formula and a Diagram for the Spacing of Stirrups (38-105)

BY E. ZBINDEN*

Tables and diagrams for the spacing of stirrups have been published before, but none of these methods has proven entirely satisfactory.

A method, formula and diagram are presented here.

The formula itself is easy enough to remember and the diagram based on it is entered with two values S and N which have to be computed in any case.

In this method, the shear triangle is divided into N unit areas. The location of each stirrup within the unit area it takes care of is such that it divides the unit area into two equal parts. This is somewhat different from the center of gravity of the unit area. Which location is theoretically

*Montreal, Que.

more nearly correct remains to be proven. The 1928 ACI Handbook located the stirrup by the equal area method, while the 1939 ACI Handbook uses the center of gravity.

In the diagram, each horizontal line corresponds to a fixed value of $S:\sqrt{N}$. For each such value, the distances a_n were computed to the nearest inch, the spacings obtained and then these spacings were rearranged in groups of equal and standard spacings. The new distances a_n thus obtained differ from those computed directly by not more than one inch except in a few isolated cases.

Another advantage of the spacing formula introduced here is that it may be useful in other problems than the spacing of stirrups. These other possibilities have, however, not been investigated.

DESIGN OF STIRRUPS

The design of stirrups entails the determination of the number of stirrups required and their spacing. While the number of stirrups depends upon the area of that part of the shear diagram for which they are required, their spacing depends upon the shape of the diagram.

Three types of shear diagrams will be considered here: rectangular, triangular and trapezoidal.

Rectangular shear diagram

After having selected the type of stirrup, its size and the steel stress, the constant spacing is given by the formula:

$$s = \frac{A_v f_v}{v' b}$$

Values of $(A_v f_v)$ are listed on the spacing diagram.

Triangular shear diagram

After having selected the type of stirrup, its size and the steel stress, the procedure is to determine:

1) The length of beam for which stirrups are required (base length of shear diagram), S.
2) The number of stirrups required, N.
3) The spacing of the stirrups.

Formulas

1) In the case of uniform loads, the value of S is as follows: If shearing stress is v at both ends of the beam: $S = \frac{v' l}{2 v}$

If shearing stress is v_1 at other end of beam: $S = \frac{v' l}{v + v_1}$

For other loadings, the value of S can be obtained from a shear diagram.

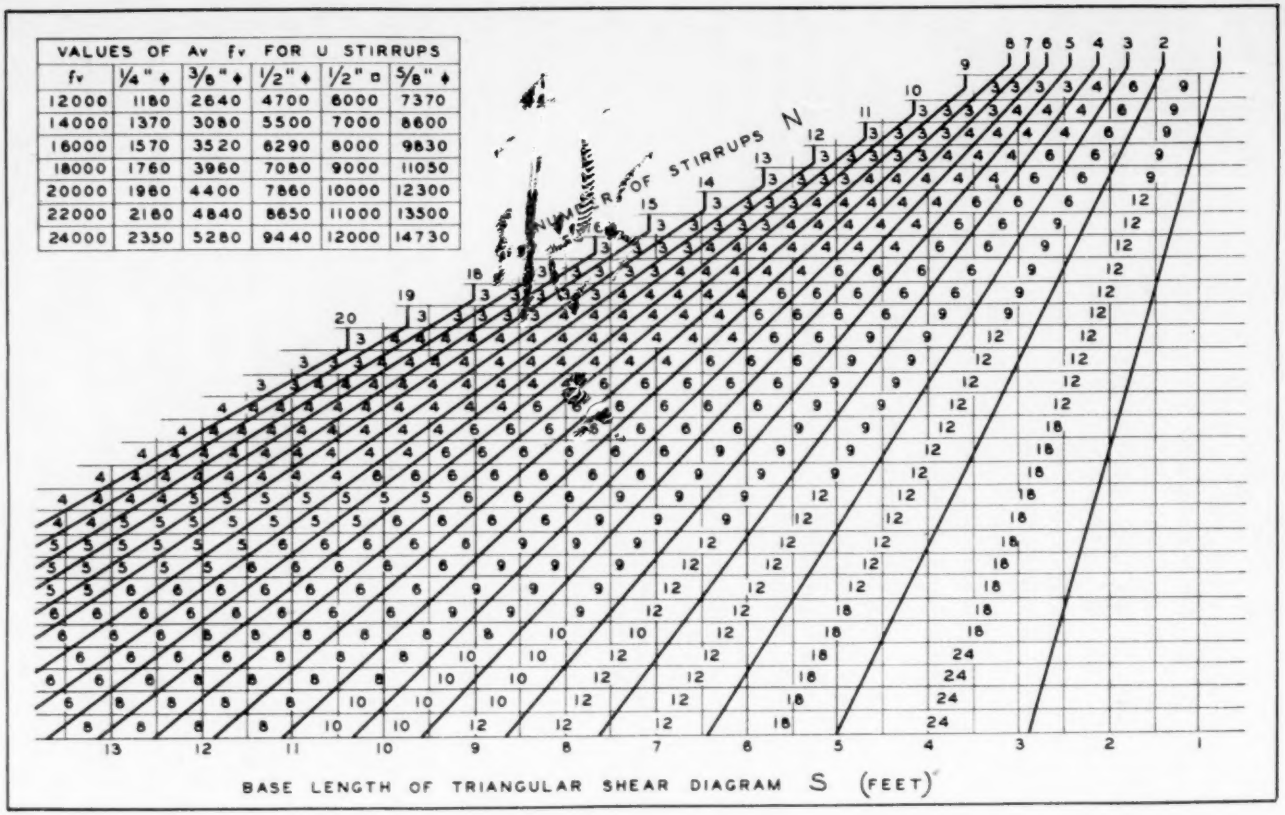

Fig. 1—Spacing of stirrups

2) Number of stirrups required: $N = \dfrac{6\,S\,v'\,b}{A_v\,f_v}$

3) The spacing is read directly from the spacing diagram for the computed values of S and N, as set forth in the examples.

Trapezoidal shear diagram

The procedure is to determine S, N and the spacing of the stirrups for a triangular shear diagram extended from and including the trapezoidal area. The number and spacing of the stirrups located within the trapezoidal area is then the correct shear reinforcement for this part.

If S_o is the base length of the trapezoidal shear diagram, in feet, we have the following formulas:

$$S = S_o \frac{\max v'}{\max v - \min v} \quad \text{or} \quad S = S_o \frac{\max V}{\max V - \min V} \frac{\max v'}{\max v}$$

$$N = 6\,S \frac{\max v'}{A_v\,f_v}\,b$$

The spacing is then taken as for a triangular shear diagram, directly from the spacing diagram, utilizing only that part located between S and S-S_o. The number of stirrups required is known once we have the spacing.

As we can determine directly the number of stirrups required in the trapezoidal area, this affords a convenient check if such is deemed necessary.

If N_o is the number of stirrups required in the trapezoidal area, we have:

$$N_o = 6\,S_o \frac{(\max v' + \min v')}{A_v\,f_v}\,b$$

General formula for the spacing of stirrups

The spacing of stirrups, in the case of a triangular shear diagram, is governed by the following formula:

$$a_n = \frac{S}{\sqrt{N}} \sqrt{n - \tfrac{1}{2}}$$

a_n is the distance of the (n)th stirrup from the apex of the shear triangle. (n) takes successively all the values from N to 1.

Note: The division of the triangular shear diagram into (N) equal areas, corresponding to (N) stirrups, is governed by the formula:

$$b_n = \frac{S}{\sqrt{N}} \sqrt{n}$$

b_n is the distance of the (n)th division line from the apex of the shear triangle.

Example 1—Triangular shear diagram.

v = 155 psi. f'_c = 2500 psi. v' = 80 psi.
l = 24.0 ft. f_v = 20,000 psi.
b = 13 in. ⅜ in. U-stirrups

From the spacing diagram, we find that for the stirrup and stress selected, $A_v f_v = 4400$.

$$S = \frac{24.0}{2}\,\frac{80}{155} = 6.2 \text{ ft.}$$

$$N = 6 \times 6.2\frac{80 \times 13}{4400} = 8.8, \text{ say 9 stirrups.}$$

On the spacing diagram, find the intersection of a vertical line at S = 6.2 ft. and the inclined line corresponding to $N = 9 + 1 = 10$ stirrups. The spacing is then read off the nearest horizontal line from the point of intersection towards the right. In our case, we read:

3 @ 4 in., 4 @ 6 in., 1 @ 9 in., 1 @ 12 in.

Note: This is the spacing of 9 stirrups within the base length of 6.2 ft., but as if a 10th stirrup were located at the face of the support. The theoretically exact inclined line to be used should be one corresponding to $N + \frac{1}{2}$, or N = 9.5 in this example.

Example 2—Trapezoidal shear diagram

v max = 175 psi. f'_c = 2500 psi. v' max = 100 psi.
v min = 110 psi. f_v = 20,000 psi. v' min = 35 psi.
S_o = 5 ft. 9 in. ⅜ in. U-stirrups $A_v f_v$ = 4400
b = 13 in.

$$S = 5.75\,\frac{100}{100 - 35} = 8.8 \text{ ft.}$$

$$N = 6 \times 8.8\frac{100 \times 13}{4400} = 16 \text{ stirrups}$$

The spacing for the triangular shear diagram is:

4 @ 3 in., 5 @ 4 in., 5 @ 6 in., 1 @ 9 in., 1 @ 12 in.

The stirrups located within the base length S_o = 5.75 ft. are:

4 @ 3 in., 5 @ 4 in., 5 @ 6 in.,

making a total of 14 stirrups, the last one being located 5 ft. 2 in. from the face of the support.

Checking the number of stirrups, we have:

$$N_o = 6 \times 5.75\frac{(100 + 35)\,13}{440} = 14 \text{ stirrups.}$$

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Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Air raid protection

Engineering News-Record, V. 128, No. 1 (Jan. 1, 1942) p. 34-35 Reviewed by S. J. CHAMBERLIN

A check list of fundamentals from two government publications, "Protective construction—Structures Series—Bulletin No. 1" and "Blackouts" (Superintendent of Documents, Washington, D. C., 25 cents each).

Simplifying concrete street construction

Engineering News-Record, V. 128, No. 9 (Feb. 26, 1942) p. 59-60. Reviewed by S. J. CHAMBERLIN

Twenty to 30-ft. pavements have plane surfaces on three slopes passing through the ordinates of a parabola. Curbs in 6-ft. lengths are placed first with removable steel separators between sections. The 12-in. gutter section has the steepest slope and is the last to be placed.

Concrete rockers for Hawaiian bridges

RUSSELL C. BRINKER, Lieut. (CEC) USNR, *Engineering News-Record*, Vol. 128, No. 11 (March 12, 1942) p. 111-112 Reviewed by S. J. CHAMBERLIN

The concrete rockers are 24 x 24 and 36 in. high. A flat bearing surface extends over about one-third the width with a ¾-in. lead plate for a cushion at top and bottom. Phosphor-bronze shear plugs are centered at both ends. The rocker is heavily reinforced, as are the super-structure and piers in the area of the rockers.

Trends in concrete highway design

A. A. Anderson, *Engineering News-Record*, V. 128, No. 10 (Feb. 12, 1942) p. 93-95.
Reviewed by S. J. CHAMBERLIN

Economy and scarcity of materials are eliminating needless refinement of design. Since steel may be reduced by improved joint spacing and proper crack control there is a renewed interest in joint-spacing research. Treated cements show promising results in reducing the effect of chemicals and frost action on pavement surfaces. Because of the lack of bleeding, treated-cement concrete requires faster finishing.

Concrete flower and plant pots

R. H. BAUMGARTEN. *Concrete Building & Concrete Products*, V. 17, No. 3, March 1942, pp. 37-39.
Reviewed by J. C. PEARSON

High price and scarcity of clay pots in England offer an excellent opportunity for products manufacturers to make concrete pots. The methods of manufacture for small

and large pots are given in detail, together with cost of production equipment, materials and labor. It is shown that concrete pots can be made and sold at prices which, compared with pre-war prices of clay pots, afford a satisfactory margin of profit.

Grouting tunnel lining on the aqueduct

Engineering News-Record, V. 128, No. 1 (Jan. 1, 1942) p. 39-41 Reviewed by S. J. CHAMBERLIN

Newer practices in concrete placing on the Delaware aqueduct result in less grouting. The grouting is done, through pipes left in the lining, with a displacement pump that forces neat cement, or sand and cement, grout into any opening outside the concrete lining. Low pressure grouting (up to 100 psi.) is allowed after one month and is primarily for filling voids around the concrete lining and to plug contraction and construction joints. High pressure grouting (up to 600 psi.) is allowed after three months.

Paving methods at an eastern airbase

Engineering News-Record, V. 128, No. 1 (Jan. 1, 1942) p. 32-33 Reviewed by S. J. CHAMBERLIN

The concrete involving almost 500,000 sq. yds. is batched in a conventional highway paver's plant and mixed and distributed in 25-ft. widths by a paver. The pavement is 6 in. with thickened edges. A power joint machine cuts 1½ in. deep dummy joints which are filled with a removable steel strip and later poured with bituminous material. Frequent movements between short runs resulted in the development of a "stone boat" for moving the finishing machines. Most of the work is with treated cement; untreated cement sections will furnish a basis for comparison.

Navy graving dock built off shore in the dry

E. C. SEIBERT, Capt. (CEC) USN, *Engineering News-Record*, V. 127, No. 25 (Dec. 18, 1941) p. 59-62 Reviewed by S. J. CHAMBERLIN

After floating dredges had excavated to a 60-ft. depth, the area was enclosed by a dike and dewatered by many small deep well pumps while well-points protected slopes. The advantages of the small pumps were: availability; low cost; decentralized flow; opportunity for servicing. After dewatering the site was graded and a gravel base prepared for the concrete floor which was placed in alternate block sections. Concrete was delivered in 8-yd. buckets from a 826-ft. span cableway supported on traveling towers. Construction of side walls was begun as soon as a section of the bottom was completed.

Putting a river underground

Engineering News-Record, V. 128, No. 5 (Jan. 29, 1942) p. 42-45. Reviewed by S. J. CHAMBERLIN

The twin 30 x 19-ft. pressure conduit will carry a 4-lane express highway on its top. A 6-in. layer of lean concrete is laid over the soft bottom, invert forms are set and filled alternately, with wall and arch placement following. Steel forms for one outside wall and two inside walls (one side of the centerline) are attached to each of two carriers. Ten 20-ft. long sets of arch forms are used for each barrel and are designed to collapse and move forward through forms in place. The concrete is mixed by a paver and pumped by a double and a single-cylinder pump through a single line to point of placing. The discharge end of the line is supported under a long boom of a crawler crane.

Falsework for arches set on drum gates

Engineering News-Record, V. 128, No. 15 (April 9, 1942) pp. 80-81 Reviewed by S. J. CHAMBERLIN

Falsework used to support the concrete arches of a highway bridge across the Grand Coulee dam spillway was framed as three-post bents with brackets to support overhanging cap ends. Two legs of the 21-ft. wide bents were carried on the steel of the

drum gates. Where they could not bear on stiffner ribs in the steel drum gate structure the load was distributed by a 4-ft. length of 10 x 10-in. steel H-beam. The placing sequence was such as to avoid unbalanced thrusts during construction. The falsework height was adjusted by wedges so that the finished barrel would be at the design elevation throughout.

Effect of admixtures on portland cement

CHAS. H. JUMPER and GEORGE KALOUSEK. *Rock Products*, V. 45, No. 4, Apr. 1942, pp. 54-57 and V. 45, No. 5, May 1942, pp. 28-29 Reviewed by ROY N. YOUNG

Aluminum, alkali, and calcium salts, a group of miscellaneous inorganic compounds, and a group of organic compounds were added in varying amounts to three cement clinkers—one high early and two standard of different compositions. Observations were made on water-cement pastes relative to condition of the paste at one minute after mixing, setting time, and temperature changes of pastes with reference to time. The admixtures had marked effects upon these properties.

Although no conclusions were drawn or basic relationships established, it is believed that such studies will assist in an understanding of the setting and hardening phenomena.

Concrete beehive for munitions storage

FRANCIS R. MACLEAY, *Engineering News-Record*, V. 128, No. 13 (March 26, 1942) p. 74-76. Reviewed by S. J. CHAMBERLIN

An elliptical concrete dome consisting of a 6-in. shell of concrete with radial 4-in. bar-trusses, which act as erection aids and support for the $\frac{3}{8}$ to $\frac{1}{4}$-in. horizontal tension wires encircling the structure, offers considerable savings in steel, concrete and labor over the conventional "igloo." Construction is to be simplified by using a polygon shape rather than a true spheroid. Steel centering and masts for the bar-trusses are designed for manual handling. To help reduce the results of explosion blasts as well as to reduce temperature and shrinkage stresses the dome would not be fastened to the concrete base.

Navy drydocks make construction history

Engineering News-Record, V. 128, No. 13 (March 26, 1942) p. 48 Reviewed by S. J. CHAMBERLIN

Underwater methods have speeded drydock construction and have set precedents for other types of work. The site is excavated below water level by dipper dredge, leveled off with I-beam drag and covered with a layer of crushed stone. Steel box forms are lowered onto steel H-piles for concreting the 14-ft. thick bottom. The concrete is conveyed by pipeline to a tremie-concrete barge where it is discharged into hoppers over the tremie pipes. Eight tremie pipes in line permit the placing over a 158-ft. width of bottom at one time with no sidewise translation nor breaking of tremie seals. Included are four attractive reproductions of drawings by Miss Lili Rethi.

White cement floors for better light

Engineering News-Record, V. 128 No. 11 (March 12, 1942) p. 88-90 Reviewed by S. J. CHAMBERLIN

In the new bomber assembly plants a $\frac{3}{8}$-in. topping consisting of a 1:2$\frac{1}{2}$ mix of white cement and white silica sand is being used on the floors to reflect light onto the underside of the wings and fuselages. Of the same mortar proportions as the concrete base course placing begins as soon as the base has hardened sufficiently to keep heel marks inappreciable. Deposited by wheelbarrows, the mortar is spread by hand shovels and struck with a wood template. Following initial set the surface is machine troweled with three rotating trowel blades. Final finish is by hand with celluloid trowels. Curing is aided by sodium-silicate spray coats protected by Kraft paper and sand. Final operation is washing and polishing.

Sliding scaffolds speed cement plant

Engineering News-Record, V. 128, No. 13 (March 26, 1942) p. 50-53 Reviewed by S. J. CHAMBERLIN

The 80 x 467-ft. raw storage building consists principally of a series of transverse concrete bents, 20 ft. apart, made up of columns along the outside wall line, crossbeams under the floor and arched roof beams carrying an arched concrete roof of 77-ft. clear span. Concrete girders connecting the bents on each side carry a crane track. At the haunches of each arched roof beam is a cross-tie made up of four steel angles supported by hangars. The interior is divided into eight storage compartments by retaining walls. Two 100-ft. traveling scaffolds, 50 ft. high, served as form falsework and working platforms for sections corresponding to lengths between expansion joints. In order to clear the tie rods all forms and supports are shifted to one scaffold and the clear scaffold moved ahead. The forms are then transferred and the second unit is brought up in line. Ordinary scaffolding is erated on the outside of each section.

Grid system floor of record spans

STEPHEN D. SZEGO, *Engineering News-Record*, V. 128, No. 1 (Jan. 1, 1942) p. 36-38
Reviewed by S. J. CHAMBERLIN

Steel columns with reinforced-concrete cantilever beams having their steel beveled and butt-welded to the column faces support a grid system of reinforced concrete beams 20 in. deep. The 4-in. slab floor is designed for 200 lb. per sq. ft. The columns are spaced 56 ft. one way and 44 ft. in the other direction with a diagonal length of about 70 ft. between column centers in the line of the beams. From the columns the 28-in. crossed cantilever beams extend to the third point of the span. The ends of the cantilevers are connected by a double set of diagonal grid beams having a spacing of 11 ft. 4 in. on centers. The principles involved consist in the equalization of deformations in two rows of statically indeterminate sets of continuous beams, the varying lengths of which results in almost complete equilibrium and causes appreciable reduction of the moments in the frame.

Absorptive form lining tests at Friant

Engineering News-Record, V. 127, No. 25 (Dec. 18, 1941) p. 72-74 Reviewed by S. J. CHAMBERLIN

Superior surfaces can be obtained without troublesome requirements or special tools when absorptive form lining is properly used and kept dry prior to concrete placement. The Bureau of Reclamation specifies values for absorption, transverse strength and deflection, tensile strength linear expansion and size tolerances. Cantilever forms are well suited to the use of absorptive form lining because of the absence of struts and ties. Form surfaces must be reasonably smooth and free from offsets before the lining is nailed in place. To avoid sticking, forms should be stripped before the concrete is 36 hrs. old. The concrete should be placed and properly vibrated in layers not more than 12 in. thick. Small vibrators may be used within 6 in. of the lining and large vibrators kept at least 12 in. away. The form lining can be used only once but the not excessive cost is partially offset by savings in cleaning and oiling of forms and the filling of surface voids.

Prestressed concrete water tanks

Engineering News-Record, V. 128, No. 17 (April 23, 1942) pp. 64-65 Reviewed by S. J. CHAMBERLIN

The tanks have an inside diameter of 106 ft. with a height to overflow of 30 ft. and a dome rise of 13½ ft. The footings were placed in sections and the floor placed in concentric circles from the outside to the center. The inner 15-in. wall was placed to the full height, with three-tenths of the periphery formed at one time. The wall forms were made of panel sections about 8 ft. square. After the concrete shell had set, 74 round

steel bands, 1⅜ in. thick, consisting of ten rods each connected by turnbuckles, were placed around the wall and prestressed by means of the turnbuckles. Forms for the 4½ in. wall covering the steel bands were 4 x 15 ft. lined with plywood. The concrete was placed in lifts of 3.8 ft., the forms being moved up as the work progressed. A dome ring bearing directly on the top of the wall, separated by an asphalt joint, extended approximately 8 in. in on the dome curve. Before placing the dome, nine steel bands were prestressed around the dome ring. The dome itself was placed in three concentric sections.

Determining the heat evolution of cement

A. TAKATA *Cement*, 1941, 14 (9), 157-60. Building Science Abstracts, V. XIV (New Series) No. 11, Nov. 1941, p. 164, (London). HIGHWAY RESEARCH ABSTRACTS

The report, which is an abridgement of the author's original paper, describes the thermos-flask method of measuring the heat evolution of cement and thence obtaining the heat of hydration. The heat dissipated is calculated as the product of the leakage constant of the thermos flask, and the temperature curve of the specimen is plotted in a diagram as a function of time. The leakage constant is obtained from the velocity of temperature decrease of hot water placed in the brass container within the flask. The procedure is described and the results obtained by this method on neat cement are shown graphically together with examples of the effect of temperature of materials before mixing and of using different water ratios at constant temperature. It is shown that the ratio between the tensile or compressive strength and velocity of heat evolution of cements is of importance in the placing of mass concrete and should be high in cement used for such purposes.

Pre-cast concrete water storage tanks

ANON. *Concrete Building & Concrete Products*, V. 17, No. 2, Feb. 1942, pp. 25-26.
Reviewed by J. C. PEARSON

Difficulties in providing reserve supplies of water for use during air raids in England have been rather serious, and various types of emergency storage have been constructed. This article describes and illustrates a simple tank of the silo type, 5 ft. deep by 16 ft. diameter, made of 25 reinforced slabs or staves, hooped with steel bands, and holding 6000 gal. Plastic jointing material is used to give watertight joints when the staves are put in compression by tightening the steel bands. An interesting scheme was adopted to overcome the difficulty of getting sufficient tension in the bands against the friction of the concrete. The slabs are tapered slightly to give a conical exterior. The bands are set only moderately tight at the top and are then driven down, like hoops on a barrel, to produce the necessary compression in the slabs. The slabs are set on pre-cast foundation blocks laid in a circle, and the bottom is a thin slab cast in place after the walls are erected.

Precast concrete framing units for navy warehouse

Engineering News-Record, V. 127, No. 25 (Dec. 18, 1941) p. 67-69 Reviewed by S. J. CHAMBERLIN

Dense concrete, accurately controlled under "factory" casting conditions, avoidance of need for shoring and greater speed in building erection, are some of the advantages secured by prefabricating the reinforced concrete second floor framing for a two story building. Actual cost is some 25 per cent under the estimate for cast-in-place construction. The struts and joists are cast on a platform, the reinforcing is welded into a cage, and the dry mix concrete is dumped on the tops of the open forms from transit mixers. The platform is vibrated, stopped, forms refilled ane vibrated. A battery of hydraulic jacks compress the forms from the side. A 1:3½ mix is used, with sand and

No. 3 granite rock proportioned for maximum density. With a water-cement ratio of 3.3 gal. per sack the 7-day strengths are as high as 6,500 psi. The units are delivered by truck and placed with a crane. The girders are cast in place with the ends of the prefabricated units projecting into the forms. Projecting steel bars from the ends of the units are welded together for continuity through the girders.

Bomb tests of materials and structures

Engineering News-Record, Vol. 128, No. 5 (Jan. 29, 1942) p. 47-49. Reviewed by S. J. CHAMBERLIN

The article gives data from the War Department on the resistance of various types of construction under actual bombing. One set of tests were made on wall panels (concrete not included, except cinder blocks) placed on a circumference around the bomb. Tests with air-raid shelters were made to determine resistance of several types. Underground reinforced-concrete box shelter provided good resistance, as did properly designed corrugated-iron structures. Sand-filled bombs dropped from 4000 to 12,000 ft. on to uncovered roof slabs of 5, 8 add 12-in. thicknesses caused local damage and perforated the slab. Indications were that the concrete and reinforcing steel acted separately and that the resistance of the reinforcing was relatively small. Small bars are better than an equal area of large bars. Under actual bombing much shattering occurred to beams and columns in building prototypes. The use of straight bars is recommended without bending. Tests on a semi-buried structure emphasized the need of rounded surfaces and an adequate roof to resist direct hits. Sand-filled and live bombs indicated that 1-ft. thick burster slabs placed over earth are inadequate.

Warehouse designed to speed construction

Engineering News-Record, V. 128, No. 17 (April 23, 1942) pp. 78-80 Reviewed by S. J. CHAMBERLIN

The 540 x 1400-ft. storage warehouse consists of 22 ft. 4 in. x 40 ft. bays, with 18-in. circular concrete columns supporting a 6-in. thick concrete roof slab, which rests on 15 x 40 in. beams running the long way of the building. Over a 52-ft. wide monitor section the 3½ in. slab is supported by inverted or overhead beams. To eliminate disposal pumps, the rain-water will be impounded in ponds on the slab roof. The floor was allowed to slope to reduce earthmoving, but the roof is level. Three 100-ft. sections of traveler and forms for the main roof slab were provided for each of the bays in the full width of the building. A traveling bridge, riding on permanent crane rails, carried the forms for the monitor roof. Truck mixers were loaded at a portable batching plant and the concrete hoisted to the roof in a low-loading height bucket where it was distributed by big-wheel concrete carts. A rich mix, six bags of cement, and finely ground cement, supplemented by preheating the materials with steam jets, produced a concrete of high early strength characteristics. Tarpaulins, salamanders and straw were used during cold weather concreting.

Concrete in sea-water

HOMER M. HADLEY, *A Revised Viewpoint Needed*, Trans. Am. Soc. C.E., V. 107 (1942) p. 345 (also *Proc.* Jan. 1941). Reviewed by H. J. GILKEY

From the results of extended visual observations on concrete structures exposed to sea-water along the Pacific Coast the author concludes that evidence of sea water (sulphate of magnesium) attack is almost wholly lacking. The paper is illustrated by cases of both good and bad concrete which has been subjected to sea-water exposure. Lack of durability is ascribed in nearly all cases to permeability or porosity from poor grading proportioning or placement, non-durable aggregate or to insufficient protection of steel reinforcement, resulting in rusting and spalling.

He recommends that the sulphate of magnesium attack theory be abandoned and that for sea-water resistant structures the attention be focused upon the need for dense impermeable concrete of sound aggregates and for adequate cover over reinforcement.

As published in the Transactions, Am. Soc. C.E., several discussions and a closure are appended to the original paper and the entire assembly constitutes a rather full coverage or Symposium upon the subject of sea water exposure for concrete. Several shades of belief are represented.

Experiments on the reduction of crack formation in road concrete

W. Schonberg, *Betonstrasse*, 1940, 15 (11), 135-7. *Road Abstracts*, Oct. 14, 1941, Vol. VIII No. 11, p. 135s
Highway Research Abstracts

For various reasons, attempts to reduce cracking in concrete slabs by using special cements, constructing dummy joints in long slabs, or employing special aggregates such as pumice, slag, etc., have met with little success. A method that has given good results is the use of burnt clay "chippings" as aggregate. The results of observations over a period of three years on an experimental surfacing of this type are as follows: (a) Shrinkage has been very small and no transverse cracks are yet apparent, (b) Some longitudinal cracks appeared during 1940 in the slabs laid on curbed sections, (c) Special measurements of surface wear were not undertaken, but the rate of wear is somewhat greater than in concrete containing hard aggregate, (d) Wear occurred evenly over the whole surface, and potholes, etc., did not develop, (e) Disintegration began at one of the transverse joints soon after the concrete was laid, but this is attributed to insufficient consolidation as the other joints remained in good condition. Reference is also made to the experimental use of pre-coated chippings as aggregate. Only one transverse crack has occurred in the series of 164-ft. slabs constructed in 1937-38 with concrete of this type. A third process that has been used experimentally on the Cologne-Wupperstal motor road consists in adding a slow-breaking bitumen emulsion to the mixing water for the concrete. It is claimed that the bituminous material both renders the concrete waterproof and reduces the shrinkage. The slabs, which were up to 123 ft. long, were placed in 1939 and showed no cracking at the end of 1940.

Stress concentrations in plates loaded over small areas

H. M. Westergaard, *Proc.* Am. Soc. C.E., Vol. 68, No. 4, Part 1 (April 1942) p. 509-534.
Reviewed by H. J. Gilkey

The paper is a mathematical analysis of the stress situations resulting from concentrations of loads on plates or slabs for which the following conditions or limitations are assumed to exist:

(a) The loaded area is not too close to any line or point of concentrated support nor to any edge.

(b) The main dimensions of the loaded area are not too small compared with the thickness of the plate or slab.

(c) The material of the slab can be considered to be homogeneous, isotropic and elastic.

The author expresses the stresses and deformations in the region of the concentration by simple formulas in terms of a set of three "place coefficients" and a set of three "area coefficients." The place coefficients depend only on the size and shape of the plate or slab, the type of support and location of the load. The area coefficients depend only on size and shape of loaded area, and on the manner of distribution of the load over that area. Symmetry may reduce one of the three place coefficients to zero.

Six sets of place coefficients are derived which number can be extended indefinitely. Twelve sets of area coefficients are derived. A numerical example is given, using a

pavement slab for illustration. The method can be modified for extension to problems in which the loaded area is small compared with thickness of plate or slab. The method can be adapted to the solution of many problems of stress concentration in plates.

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Discussion of Report of Committee 611:

ACI Manual of Concrete Inspection*

DISCUSSION BY A. BURTON COHEN, ROY W. CRUM, O. G. JULIAN, H. F. THOMSON, W. R. WAUGH, HERMAN G. PROTZE, JR., with FREDERICK B. PARKS and DONALD G. BALL, A. T. GOLDBECK, H. G. FARMER, WILFRID SCHNARR, M. A. SWAYZE—Closing remarks by J. W. KELLY, Chairman Committee 611

Discussion was voluminous and detailed; parts of it occupied an entire session at the Convention, February 1942. The usual procedure in recording each discussion fully, in one piece, is abandoned in favor of reader convenience, by breaking up each discussor's contribution into parts applicable to chapters and sections of the Manual and grouping these parts in the order of the Manual's subject matter, so that each part of the book is considered in one sequence. This arrangement is preceded by the discussors' general comments on the importance of concrete inspection and on the Manual, its purpose and treatment as a whole.—Editor.

THE MANUAL AND INSPECTION IN GENERAL

A. Burton Cohen:† I think the Committee has done a very good job in developing this Manual, but I should like to emphasize the fact that the Manual is of little value if it is not put into the hands of men who understand what it is all about, and the basic principles that control the development of good concrete. So this evening I am not concerned so much with the rules and regulations that have been developed to control concrete construction, but I am concerned about the ability of the engineer to direct the regulations. This most important phase of producing a sound structure has been woefully neglected. Research men and designers have spent years and years to improve the art but their efforts are wasted by poor supervision in the field.

"Inspection" an inadequate term

You will note that I have avoided the term inspection. I think it is an inadequate term; the results obtained are commensurable with the

*Issued as a special publication (141 pages) to ACI members, July 1941.
†Consulting engineer, New York, N. Y.

meaning of that word; how can one direct or properly interpret the intent of the design and specifications if he is not familiar with the fundamental principles of design and has not followed the work of the research men to know the significance and the underlying principles which govern the formation of the rules and regulations?

We need technically trained men or their equivalent; there is a fine field for the college man with the right aptitude and force. They must be compensated in accordance with their knowledge and the importance of their responsibility. The so-called inspector must be replaced by an engineer of construction, technically trained. This is a job that cannot be filled by any college graduate. It is an easy matter to determine whether or not a man has the right temperament or a student has the right temperament or ability for this important field which leads to the control of the industry. The technical school is the first best medium to tackle this problem. Other agencies and industry can put on the finishing touches. An engineer who is trained to start as an engineer-inspector should advance rapidly in the construction field to the position of superintendent. Good superintendents are scarce; so there is a field for the college man to go from this position of engineer of the work to that of superintendent and on to the contracting field. As I listen to our discussions in the Institute and think of the fine work that is being done by the research man and by those who present improvements in design, there runs in my mind a certain futility of effort because we are not paying sufficient attention to carry out the principles of the fine work that these men are doing. It has been my experience, and I am not talking about all agencies of construction of all work but in general, that inspection is something to worry about; there are many instances of absolute ignorance of inspectors in my memory but I shall not attempt to relate them. It is so important however to find men of ability or to develop men of ability who can carry on the work that has been advanced by this Committee and to carry it out with an intelligence that will produce results.

More discussion of human relations in inspection

*R. W. Crum:** Careful perusal of this Manual impresses one with the fact that it is a very workable document and that if followed thoughtfully good concrete should result. Mr. Kelly and the Committee are to be congratulated on a fine piece of work which hits so closely to the bullseye of their objective.

Of course nothing is perfect on the first attempt, nor on any attempt for that matter. But everything can be improved although perfection may be unattainable, and since it is expected that the Manual after

*Director Highway Research Board, Washington, D. C.

being put through the refining processes of study and revision will become a Standard of the Institute, it is desirable for those interested in the subject to set down for the benefit of the Committee such constructive criticisms as they have, no matter how small in import. This is my excuse for observations to follow.

I realize that consideration was given to the amount of material to be included bearing upon the relations of the inspector to the contractor on the one hand and to the owner on the other, and to the human factors of judgment and discretion. However, since these factors loom so large in the every day problems of the inspector and since competency in handling human relations and in making allowance for the always necessary tolerances in administering construction specifications is so vital to success, I should like to see some extension of the discussion of these factors in the Manual.

*O. G. Julian:** Committee 611 should be commended upon turning out a splendid piece of work. Everyone responsible for the installation of concrete, whether it be on a small job or in connection with monumental structures, should be familiar with the contents of the Manual. The term "everyone" as here used includes engineer, contractor, superintendent, foreman, and inspector. Unless this personnel is working in close harmony and has sympathetic viewpoints, "harshness" on the job is likely. This harshness is no more conducive to good concrete than is harshness of the mix.

The Manual has been published in convenient pocket size—unfortunately, in paper covers which are not durable.

Trained technical personnel needed

It should be emphasized (as it is in the Manual) that concrete is not simply a more or less arbitrary mix of cement, stone, sand and water, mixed until it looks uniform, and then "poured" into forms without benefit of adequate working, protection and curing. The manufacture, installation and curing of concrete requires trained technical personnel on all parts of the job and this personnel should be supplemented by the facilities of an adequately equipped and staffed testing laboratory. No one can tell by his unaided senses if the component parts or the mixed product will produce good, durable concrete. One might as well attempt to manufacture steel without the aid of trained supervision, chemists, metallurgists and adequate testing facilities as to manufacture concrete without corresponding personnel and facilities. The working and curing of concrete are analagous to the working and annealing of steel. The former, however, is too often left to a foreman who has not read the specifications, is totally unsympathetic with their require-

*Jackson & Moreland, Engineers, Boston, Mass.

ments (because they are different from what he has been doing in the past) and who is full of excuses for not working and curing the concrete. These excuses apply especially to the curing of vertical and overhead surfaces. On the other hand, steel is made, worked and annealed under the strict control of trained technicians. The difference in supervision is directly reflected in the uniformity and reliability of the finished product. As stated by the late George F. Swain,* concrete may be an unreliable and not a uniform product; if this is so it is solely because those who manufacture, "pour" and do *not* cure it have not been properly educated. That concrete need not be poured like pea soup but should be worked into place with tampers, spades, etc. was well known in the last century; but this fact and the proper use of more recently introduced vibrators is a field of education unto itself. The distribution of the ACI Manual should do much to further greatly needed education. It should be "required reading" for all structural engineering students, concrete material men, concrete manufacturers and contractors, and others who have anything to do with concrete design and installation.

It is unfortunate that in many cases the manual has referred to "minimum specifications" without also giving references to more stringent requirements in current use. (Examples will be mentioned in their appropriate places in the following pages.)

H. F. Thomson:† The committee who compiled the Manual are to be commended highly. The scope of the material is excellent, and the thorough, and at the same time, concise, treatment is noteworthy. It meets a need among concrete technicians which no other publication has supplied. We have found that this little volume serves to give inspectors for public agencies, especially the newer men, a further background for the "why of concrete" than they have received from their departmental indoctrination. And we have also distributed copies to certain concrete contractors who have found that much of the contents, such as the excellent chapter on "Fundamentals of Concrete" supplements and clarifies many of their rule-of-thumb conceptions.

W. R. Waugh:‡ The Manual is a valuable contribution to concrete literature, and is an excellent beginning on the answer to a distinct need. Every agency engaged in concrete construction, whether private or public, has, no doubt, felt the need for just such a manual, and many agencies have prepared and published manuals to suit their own needs.

The first line of the second paragraph of the Foreword implies that the Manual is primarily intended for the inspector. The writer intends in the following to discuss the Manual from the viewpoint of a super-

Journal Boston Society of Civil Engineers, Jan. 1927.
†Vice-president General Material Co., St. Louis, Mo.
‡Materials Engineer, Hiwassee Projects, Tennessee Valley Authority.

visor who has under his direct or very close indirect supervision a moderately large group of inspectors. The inspection work is all related in one way or another to dam construction. The concrete work encompasses a range from appurtenant concrete structures for earth dams, such as intake structures, to mass concrete for gravity type concrete dams; and, the inspection work covers a wide range also, extending from inspection of concrete aggregate through all of the processes of concrete mixing, transporting, placing, finishing, cleanup, curing and protection.

Inspectors have to be "made" and need more detailed information

The present emergency has complicated the inspection problem in a number of ways, but the most serious problem which it presents at present is the scarcity of capable inspectors. A few years back capable and experienced inspectors could be chosen from a long list of applicants; today inspectors have to be "made," and that presents the problem of inspector training. In this connection the Manual in its present form can be and will be used to advantage, but the writer believes that it would be of greater value if it were more in detail and more specific in a number of respects. After the inspector has received his training he needs a ready reference, and in this instance, also, the Manual would be of greater value if it were in more detail and more specific.

Throughout the Manual there are numerous references to other publications for "additional details" or "recommended practice". Obviously this is absolutely necessary because all of the data included in all of the references would combine into several volumes. In many instances the "meat" of a particular subject is given in the Manual, and reference to another publication is made for additional details. (See specific references at appropriate places in this discussion.)

As pointed out in the Foreword, differences of opinion are bound to exist as to what should and should not be included in a manual of this kind. The member who argues for a shorter treatment no doubt reasons that it is useless repetition to include details which may be found in other publications, but this argument fails to take into account the fact that the inspector may not, and usually does not, have access to these other publications. The writer, therefore, believes that to be of greatest benefit to the inspector the treatment should be more in detail and more specific. This means that the Manual should be a complete compilation which gives the inspector not only the "how" and "why" but to a very great degree also the "recommended practice". References to source of material for greater detail is very desirable, but the minimum treatment should give the fundamental facts, and the inspector should only have to go to a reference, if his curiosity, or some

special requirements of his work sends him there for details which explain or support the fundamental facts.

*Herman G. Protze, Jr., Frederick B. Parks, Donald G. Ball:** The publication of the ACI Manual adds to the literature a much needed treatise dealing with the homelier philosophies and fundamentals of this controllable construction material. So that this handbook may become even more valuable as a guiding text for the new inspector, and as a reference check list for the senior technician, the writers present the discussion dealing specifically with sections of the Manual relating to Concrete Control. (The numerous suggestions of Protze, *et al*, appear in their appropriate places.)

INTRODUCTION—Manual p. 1-5

Messrs. Protze, Parks and Ball suggest: adding to paragraph 1, p. 1: "Often the actual expenditure on concrete control is more than balanced by savings in the cement used, and by the greater ease and economy obtained in placement;" rewriting line 17 of p. 4: "Hot plate, oven, or alcohol stove for drying aggregates; or other equipment for measuring moisture content of aggregates such as is described below;" and adding (p. 6, para. 5, l. 6): "or which will cause improper construction relative to the intent of the specifications."

FUNDAMENTALS OF CONCRETE—Manual p. 10-20

Mr. Waugh refers to p. 10, "where in the first paragraph the reader is referred to References 3, 4, 5, 7 and 58, for detail study of principles and practices; however, in the following pages (p. 10-20) the "meat" of principles and practices is given. The writer is in agreement with this method of handling, but even in this instance there seems room for more detail."

Mr. Thomson: The term, "water-cement ratio" might well be clarified. As used at the bottom of p. 12 and also shown in Fig. 5, it is applied as "ratio by weight", which is of course correct; but for practical purposes, the water content expressed as "gallons per sack of cement" is used much more commonly and is more readily understood because of the usual measurement of water by gallons instead of by pounds. The relation between the two expressions as referring to the same factor, should be pointed out. This relation is implied in the note to the caption of Fig. 10 but is not definitely stated.

Messrs. Protze, Parks and Ball: Water-cement ratios should also be expressed in terms of both volumes and gallons per sack of cement to make Fig. 5 more generally valuable. Curves "A", "B" and "J" from

*The Thompson & Lichtner Co., Boston, Mass.

Fig. 1 (Reference 3) might well be superimposed on this diagram to make its use more valuable particularly since "Abrams' Curve" is nowhere given in the Manual.

Design of Mixes, p. 16-20*

Mr. Crum: I should like to see this section go a little further and discuss the translation of the results of mix design into the familiar specification terms of 1:2:4, etc. Since nowadays the inspector may meet the statement of specified proportions in terms of volume, weight or absolute volumes, some further discussion of the interrelationships of these different methods of description should be helpful. For best practice it is true that proportions are necessarily arrived at through the medium of absolute volumes, but unfortunately the materials cannot be measured that way and for successful conversion into volumetric or weight measurement thorough understanding of the relationships involved are necessary. It is true that these matters are taken up in the section on "Proportioning", but this seems to be a case in which some repetition in thought if not in wording would not be amiss.

The Manual rightly emphasizes the desirability of arranging proportions in most cases so that a minimum of cement paste is used. Nevertheless the use of slightly oversanded mixtures has its place and should be discussed. Good engineering sometimes indicates substantial economy in the use of such mixtures and at other times to take advantage of their very real benefits is good practice. For instance under some conditions difficulties incident to gradation of coarse aggregates and variable yield may thus be obviated.

A. T. Goldbeck:† The subject matter of the Section on Design of Mixes is built up on the basis of certain fundamental conceptions which will be examined in the present analysis. It is stated, "The first consideration in mix design should pertain to the cement paste, that is, it is necessary to select the type of cement to be used and the water-cement ratio." At the present time there are different classes of cement and for that matter there always have been cements of different characteristics and so naturally the characteristics of the cement must be considered. The conception that one must select a definite water-cement ratio is based on the assumption that a definite water-cement ratio controls both the strength and the durability of concrete.

The assumption that water-cement ratio controls the strength of the concrete is founded on the general water-cement ratio strength relationship originally stated by D. A. Abrams in his Bulletin 1 of the Lewis

*Mr. Goldbeck who considers "Design of Mixes" comprehensively following Mr. Crum's suggestions, presents his discussion as applicable also to the report of Com. 613, Recommended Practice for the Design of Concrete Mixes, ACI JOURNAL, Jan. 1942; *Proceedings* V. 38, p. 193.

†Engineering Director National Crushed Stone Association, Washington, D. C.

Institute. Concrete must be mixed to a given consistency or plasticity for a given type of structure so that it may be placed and finished properly in a manner such that uniformity and freedom from honeycomb will result. Under the conception of a constant water-cement ratio it very naturally follows that aggregates which are rough and angular will require more paste than aggregates which are smooth and rounded, and, consequently, it further follows that under this conception of concrete design more cement must be used for angular, rough aggregates than for rounded, smooth aggregates, irrespective of what the final properties of these different concretes may be and entirely irrespective of the relative suitability of these respective concretes for the actual work they must perform after they are in the structure.

It becomes pertinent to inquire into the adequacy of such a conception of design to produce concrete having the desired strength and durability for the conditions under which it must serve. Years ago the admirable work done under the direction of Abrams at the Lewis Institute established the well known water-cement ratio compressive strength relationship. The average curve showing that relationship, appearing as Fig. 1 in Bulletin 1 of the Lewis Institute seems to show a very definite relationship indeed, especially when the entire curve is viewed, with water-ratios extending all the way from approximately 0.4 up to 4.0 by volume. But I wonder how many concrete technicians have stopped to think that, as a rule, they are concerned with only a very limited portion of that curve, that portion extending from 0.5 to 1.25 water-cement ratio. The curve is merely the average of a number of individual strength results and the significant portion is reproduced in Fig. A.

It will be seen from Fig. A that there is rather a wide deviation of the individual results from the average. For illustration, for a value of 6 gal. of water per sack of cement, the range of compressive strengths was from approximately 2300 up to 3600 psi., the top value being about 56 per cent higher than the low value. These are laboratory results obtained under very carefully controlled conditions and by expert technologists. The cement and aggregates were identical for all of the tests, but were mixed in different proportions. When different aggregates were used it is found that each aggregate may have a water-cement ratio strength relationship of its own, more or less parallel with a similar relationship shown by another combination of aggregates. To obtain a given strength with one set of aggregates, a given water-cement ratio must be used, but another water-cement ratio is necessary for another set of aggregates. Professor Abrams recognized this to be the case in his original design bulletin, for his first conclusion was, "*With given concrete materials and conditions of test* the quantity of mixing water used determines the strength of the concrete so long as the mix

is of workable plasticity." Please note the words, "with given concrete materials". The fact is, that a general water-cement ratio strength relationship is merely the average of a number of such relationships which differ from one another depending upon the aggregates used and their combinations.

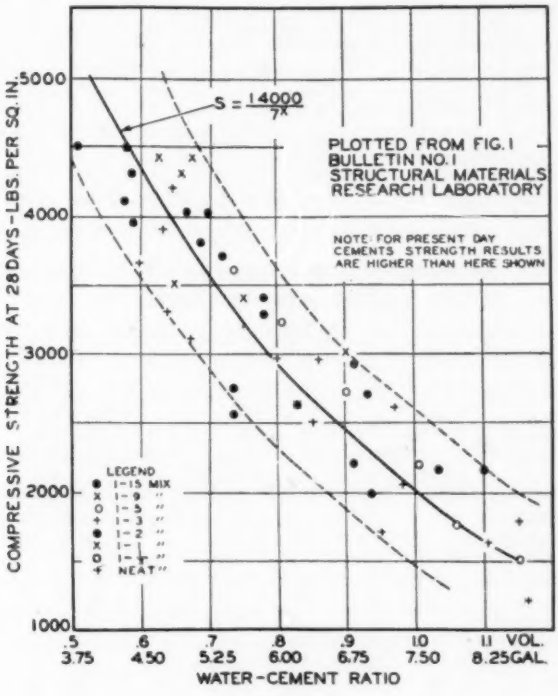

Fig. A

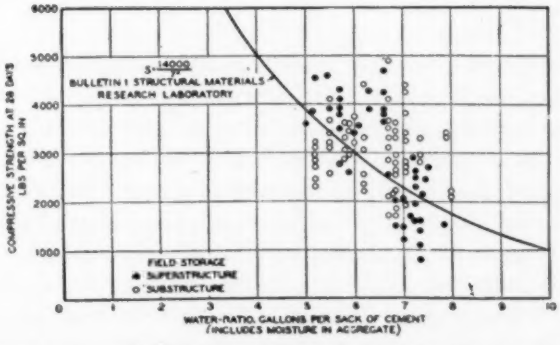

Fig. B—Compressive strength—water-cement ratio relationship from field specimens, Portland Cement Assn. Bldg., Chicago

TABLE A—COMPARISON OF CONCRETES MADE WITH A LIMESTONE AND A SILICEOUS GRAVEL

No. of Sacks	Gravel				Limestone				Stone-Gravel Strength Ratios
	b/b_o	Gallons per Sack	Total Weight of Water	Strength Compressive Modulus of Rupture	b/b_o	Gallons per Sack	Total Weight of Water	Strength Compressive Modulus of Rupture	
5	.709	6.53	272	3780 571	.703	7.25	302	3650 677	.965 1.19
6	.704	5.48	272	4390 655	.696	6.00	298	4520 720	1.03 1.10
7	.712	4.82	282	5140 707	.706	5.28	308	5440 825	1.06 1.17

TABLE B—EFFECT OF 100 CYCLES OF FREEZING AND THAWING ON CONCRETES, AGE 10 YEARS

Cement Factor, 6.3 Sacks per Cu. Yd., Freezing at —10 to —20 F., Thawing at 70 F in Water

No.	Coarse Aggregate	Water-Cement Ratio by Volume	Per Cent Loss in Weight	Permanent Expansion Inches per Inch No. x 10^{-6}	Per Cent Reduction in Modulus of Rupture
2	Stone	.79	0.0	114	51
33	Gravel	.79	0.1	295	77
5	Stone	.81	0.3	12	4
6	Gravel	.74	0.2	25	14
7	Stone	.81	0.1	87	40
8	Gravel	.75	0.0	321	66
13	Stone	.79	0.0	75	37
17	Gravel	.69	3.3	159	64
23	Stone	.78	0.2	134	61
24	Gravel	.71	0.0	273	61
117	Stone	.79	0.3	150	55
31	Gravel	.73	0.1	93	51

Actually, there are a number of factors which control the compressive strength of concrete of which the water-cement ratio, although of high importance, is not the sole factor. One merely has to examine the water-cement ratio strength relationship curve obtained with concrete made in the field to be satisfied of the truth of that statement. In Fig. B is shown such a curve obtained on field concrete mixed and tested under expert supervision. The curve was published first in *Engineering News-Record*, V. 96, No. 17 and it applies to the concrete used in the construction of the present Portland Cement Association building in Chicago. Such a curve merely demonstrates the great difficulty of controlling all of the variables which affect the strength of concrete and

it demonstrates the entire inadequacy of a given water-cement ratio as a means for controlling concrete strengths. Yet many a really superior coarse aggregate has been entirely eliminated from use, not by the application of the water-cement ratio strength relationship itself, but by an entire failure to realize its limitations. The relationship exists but it is a different relationship for different aggregates and different other conditions.

Let us be more specific by considering a comparison of two concretes, one made with a good grade of limestone and the other made with a good grade of siliceous gravel. Both materials were graded essentially alike and had a 1½ inch maximum size. Both materials were designed to have essentially the same workability. The comparison of these respective concretes is shown in Table A.

Table A shows that the limestone concretes were surely not inferior to the gravel concretes in compressive strengths and they certainly were superior to the gravel concretes in beam strengths, in spite of higher water ratios being used in the limestone than in the gravel concretes. The cement factors were the same for both aggregates. Surely, the use of some arbitrarily selected water-cement ratio, on the assumption that it would give like strengths would not be warranted in this case and if these respective concretes were designed on that basis, more cement would be required for the limestone concrete than for the gravel concrete no matter whether it needed it or not. Were these two aggregates in competition a very grave injustice would be done and, in effect, a monopoly would be created for the gravel aggregate by the method of design suggested in the Manual.

Another of the concepts evidently used in the Manual in connection with design of concrete is that it is quite necessary to keep the water-cement ratio in the paste as low as possible, because thereby less shrinkage of the concrete results, and for the same reason it is stated that the paste content itself should be kept to a minimum. It is perfectly true that with a given cement content the higher the water-ratio, the higher will be the shrinkage. But consider the cases given above in Table A. In the first place, the difference in water-cement ratio is not great as between the stone and gravel concretes and, accordingly, the difference in shrinkage would not be great. Furthermore, amount of shrinkage is only one of the factors to be considered in connection with any deleterious effects shrinkage may have. Another important factor is the tensile resistance of the concrete. If concrete were unrestrained, shrinkage would produce no internal stress. If the concrete is restrained then shrinkage can produce internal tension. But it does not necessarily follow that increased shrinkage will produce increased cracking or even wider cracking, for the concrete having that increased shrinkage may

be better able to withstand the tendency toward shrinkage because of its higher tensile resistance.

It is seen in the above examples that the limestone concrete had higher beam resistance than the gravel concrete. Beam resistance is at least a measure of tensile resistance. Accordingly, the stone concretes above cited, although possibly, but not assuredly, having a slightly greater tendency to shrink than the gravel concretes, at the same time have higher resistance to cracking due to that shrinkage. Thus, from the standpoint of possible deleterious effects from increased water-ratio, the slight difference in water-ratio as between stone and gravel concretes with the same cement factor is hardly worth a second thought.

Can this slight increase in water-ratio have a deleterious effect so far as lack of durability is concerned? It is true the paste will be just a little bit weaker and a little bit less dense, but only a little bit and there are factors which bring about lack of durability in concrete which far outweigh any slight differences in durability due to these rather inconsequential differences in water-cement ratio. By and large, it is very true that with the same aggregates a high water-cement ratio would produce weaker and less durable concrete than a low water-cement ratio and there can be no objection to the setting up of a definite maximum water-cement ratio for the purpose of attempting to control durability of concrete. But that is quite a different matter from the use of a definite water-cement ratio from which there can be no departure and because of which more cement must be used in one concrete than in another.

When different aggregates are used in concrete it is entirely possible to have a concrete of low water cement ratio showing less durability and higher volume change than concrete having a high water-cement ratio. In Table B are given some test results which will substantiate that statement.

All of the concretes in Table B were designed to have 6.3 sacks of cement per cu. yd. They are the concretes described in the National Crushed Stone Association Bulletin No. 7, "Investigations in the Proportioning of Concrete for Highways", September 1931. The broken beams from these concretes have been continuously stored in the moist room for the past ten years. From them slabs have been sawed 2 in. thick and 6 in. square. These slabs have been subjected to freezing and thawing tests up to 100 cycles, at the end of which the loss in weight was obtained as well as the permanent expansion of the specimens and likewise these slabs were tested for modulus of rupture. Time does not permit of a complete analysis of all of these data, but let it be pointed out that specimen No. 17 having the lowest water-cement ratio of all the concretes shown, has the highest percentage of loss in

weight after 100 cycles of freezing and thawing and also shows next to the largest reduction in beam strength due to freezing and thawing. This concrete was made with a siliceous gravel from Texas. The failure from freezing and thawing was classed as a mortar failure. Concrete No. 5, made with trap rock, had the highest water-cement ratio and it will be observed that this concrete suffered very little reduction in slab strength after 100 cycles of freezing and thawing and showed very little permanent expansion due to freezing, the type of failure being classified as a mortar failure.

The range of water-ratios shown in the table is representative of the range to be expected in practice for highway concretes. A study of this table will demonstrate that the effect of freezing and thawing on these concretes, whether that effect be reduction in modulus of rupture, the production of permanent expansion, or loss of weight, is not a relationship consistent with the values for water-cement ratio. Quite evidently there are other factors acting to bring about deterioration of the concrete which are more significant than the slight difference in water-cement ratio necessary for equal consistency in the stone or gravel concretes. Yet, the method of design in the manual lays the greatest stress on the water-cement ratio of the cement paste to the utter neglect of more important factors controlling the durability of the concrete.

Thus, again, it is emphasized that if concrete is to be designed for a structure on the basis of that concrete being capable of serving in that particular structure over long periods of time, something more is needed than the mere setting up of some arbitrary water-cement ratio as a means for controlling durability and then manipulating the proportions for the purpose of obtaining plasticity. The fact that an arbitrary selection of water-cement ratio irrespective of other influencing factors does not necessarily lead to the same strength or to the same durability does not mean that the water-cement ratio strength relationship does not have value. That is far from the case. It does have value, but it should not be used as a fixed value in such a way that it creates a monopoly for one type of aggregate and thereby excludes aggregates which may produce a more durable structure by a very wide margin. Thus, the use of the water-cement ratio method of design suggested in the Manual for the purpose of obtaining durable concrete of sufficient strength not infrequently defeats its own purpose. It may actually prevent the use of the most suitable aggregate.

The question now arises, What method of design should be used in lieu of the one indicated in the Manual against which such serious objection is here raised? First, the most certain method is by the use of thorough investigation of the aggregates and concrete using every known test which may have a bearing on the particular problem in hand. It

is not enough to decide on some arbitrary water-cement ratio and then merely design the concrete for proper workability. There is something far more important than mere ability to place the concrete properly. That something is the ability of the concrete to resist whatever forces and actions it may be subjected to after it is in the structure. The tests should simulate those actions and they may include freezing and thawing, expansion and contraction due to moisture and temperature, chemical effects and any others that may be important in connection with the particular structure under consideration. Beam tests should replace compression tests for road slabs.

I realize that this is a big order and frequently cannot be undertaken except in the very largest of structures and so the question arises as to what is the next best thing to do.

1. I would propose that the water-cement ratio relationship be recognized to the extent of setting up maximum values for water-cement ratio, depending on the exposure conditions of the structure.

2. With a water-ratio not in excess of that required for the desired durability and consistency a cement factor should be used which will give the required strength. This same cement factor should be used in designing concrete for different aggregates. This will require a varying water-cement ratio as well as a varying sand factor to maintain the desired consistency. In no case should the desired maximum water-cement ratio be exceeded.

The advantages of this method are:

1. A definite cement factor will be required and this is of advantage in safeguarding the quality of the concrete mixed to a given consistency.

2. The water-cement ratio principle is recognized, but not to such an extreme extent that competition between different kinds of coarse aggregates is impossible.

3. The engineer is given a wider choice of aggregates, some of which may make superior concrete, but which are virtually eliminated if a single, average water-content relation is followed blindly.

The method suggested is not new. It is recognized and used by some very important government departments and state highway departments and is the basis for Alternate B specifications in the Joint Committee Report. Its philosophy differs from that of the present chapter on design in the Manual. It is impossible merely to revise the present chapter on Design of Mixes in the Manual by the use of slight revisions; it must be re-written. There is little use at the moment in bothering with the detailed wording necessary to revise the Manual. That requires quiet study. The important thing now is to recognize that the present method of design is inadequate and does not lead to the best and most durable concrete.

Concrete may be very easily designed in accordance with these ideas by the use of Tables C and D which are based on investigations by the National Crushed Stone Association. The method of using these tables for concrete design will be apparent to concrete technicians. Their detailed use, however, is described in the literature of the National Crushed Stone Association.

TABLE C—DRY, RODDED VOLUME OF COARSE AGGREGATE (ANY TYPE) PER UNIT VOLUME OF CONCRETE (b/b_o)

Size of Coarse Aggregate, Square Opening Laboratory Sieves	Fine Sand		Medium Sand			Coarse Sand		
	Fineness Modulus of Sand							
	2.40	2.50	2.60	2.70	2.80	2.90	3.00	3.10
	Values for b/b_o							
No. 4 to ¾ in.	.71	.70	.69	.68	.67	.66	.65	.64
No. 4 to 1 in.	.72	.71	.70	.69	.68	.67	.66	.65
No. 4 to 1½ in.	.74	.73	.72	.71	.70	.69	.68	.67
No. 4 to 2 in.	.76	.75	.74	.73	.72	.71	.70	.69
No. 4 to 2½ in.	.78	.77	.76	.75	.74	.73	.72	.71

b/b_o = dry, rodded volume of coarse aggregate per unit volume of concrete.
b_o = solid volume of coarse aggregate per unit of coarse aggregate.
b = solid volume of coarse aggregate per unit volume of concrete.

Note: For concrete pavements or other work in which there should be less mortar than required in the usual structural concrete, increase tabulated values of b/b_o approximately 10 per cent.

For concrete especially designed to be compacted by internal vibration under very rigid inspection, increase tabulated values of b/b_o approximately 15 per cent.

TABLE D—CEMENT FACTORS (SACKS OF CEMENT PER CU. YD. OF CONCRETE) REQUIRED FOR 28-DAY COMPRESSIVE STRENGTHS LISTED

Size of Coarse Aggregate			No. 4 to ¾ in.		No. 4 to 1 in.		No. 4 to 1½ in.		No. 4 to 2 in.		No. 4 to 2½ in.	
Slump (in.)			3	6	3	6	3	6	3	6	3	6
Water, gal. per Cu. Yd. of Concrete	Angular Coarse Aggregate		40	42	38	40	36	38	35	37	34	36
	Rounded Coarse Aggregate		36	38	34.5	36.5	33	35	32	34	31	33
28 Day Compressive Strength	2000	Sacks of Cement per cu. yd. of Concrete	4.6	4.9	4.4	4.6	4.2	4.4	4.1	4.3	4.0	4.2
	2500		5.0	5.3	4.8	5.0	4.5	4.8	4.4	4.6	4.3	4.5
	3000		5.5	5.8	5.2	5.5	4.9	5.2	4.8	5.1	4.7	4.9
	3500		6.0	6.3	5.7	6.0	5.4	5.7	5.2	5.5	5.1	5.4
	4000		6.5	6.8	6.1	6.5	5.8	6.1	5.7	6.0	5.5	5.8
	4500		7.1	7.4	6.7	7.1	6.3	6.7	6.2	6.5	6.0	6.3
	5000		7.7	8.1	7.3	7.7	6.9	7.3	6.8	7.1	6.5	6.9

Note: The 28-day compressive strengths shown are the minimum values to be expected and should be used for design purposes. Laboratory specimens cured under ideal conditions will generally have higher strengths.

Messrs. Protze, Park and Ball suggest that para. 1, p. 18 be rewritten for emphasis on the importance of selecting the *source* of aggregate with respect to inherent strength and soundness where weather exposure of the concrete is expected, and add that aggregate from the chosen source should also be clean and well graded.

INSPECTION AND TESTING OF MATERIALS—Manual, p. 21-46

Mr. Protze and associates suggest (p. 21, para. 2) that the five types of cement should be tabulated by name and the specific use of each briefly described. Physical and chemical limits of specifications C150-41 should be given either in tabular form for all types or at least in paragraph form for the Type I cement described. A short explanation of the purpose of each test should be given so that the inspector will have better elementary appreciation of the material he is using.

They would add to para. 3, p. 21 "Color of cement received should be compared with a standard color sample (previously obtained from the accepted bin) particularly at batching plants where more than one brand is in use at a time. This will help assure the use of the correct cement and prevent undesirable changes in the color of the concrete in the structure."

Mr. Julian (who asks for references to some of the specification requirements more stringent than those mentioned) suggests (p. 21): According to a "Report of Special Subcommittee of Committee C-1 on Cement," A. S. T. M. *Bulletin* Jan. 1940, p. 39, type i cement should not be used on other than minor and unimportant work, where superior weather resistance, low heat of hydration, etc. are not required. Also the autoclave volume change is often restricted to 0.15 per cent instead of the 0.50 per cent given in the A. S. T. M. specifications C 150."

Sampling and testing (cement), p. 21

Mr. Protze suggests several text revisions (on p. 22): in line 3, "Usually one sample is taken from *each car* or for each 400 bbl . . .;" end of para. 1—"In identifying the cement sampled, the following items should also be included: name of project, cement car number, inspector's name;" para. 2, line 1: "If the cement as received *is excessively hot* (*over 125 F.*), the cause . . . "; para. 3, line 6: "the materials or concrete, by adding the hot water directly to the cement before it reaches the aggregates, or by the use of an accelerating admixture such as calcium chloride;" para. 4, line 2: "or due to an undesirable chemical reaction between certain combinations of a cement and given admixtures."

Batching (cement), p. 23

The same discussion would rewrite the first part para. 4, p. 24: "Bulk cement should only be batched by weight, usually within a specified

tolerance of 1 per cent; volume measurement should never be allowed due to extreme variations in bulking under varying conditions of moisture, fineness, compaction, etc. The inspection of weighing . . . "

Mr. Crum: The Manual (p. 24, para. 2) prescribes that the salvage from bags should either be batched with a relatively large proportion of fresh cement or should be used in unimportant work. If, as implied here, this cement is inferior in quality there might be a question as to whether its use should be recommended at all. Some years ago after a thorough series of tests I came to the conclusion that, if reclaimed cement when properly sampled passed the requirement of the standard tests, there was no reason for discrimination against its use.

Specifications (aggregates), p. 24

Mr. Julian (again for reference to more rigid specifications) mentions: "The Joint Committee requirement that the fineness modulus of a shipment of fine aggregate shall not vary more than 0.20 either way from the fineness modulus of a preliminary sample. It should be noted that the Joint Committee's Recommended Practice advocates more stringent requirements on the gradation of aggregate than is given in their Specifications and that the U. S. Bureau of Reclamation (Concrete Manual, 2nd Edition, p. 22) advocates a corresponding variation in the fineness modulus of but 0.10.

"In many cases it is desirable to use several sizes each of small and large aggregates, which are later combined in the mix to give more nearly ideal gradation. This point, although mentioned, does not appear to be sufficiently emphasized."

Mr. Crum: I had it in mind to suggest that in the sections on tests of aggregates (p. 24) and tests of concrete (p. 84) more attention be given to presentation of the significance of the various tests. But I realize that this would take a large amount of space and probably it is beyond the scope of a Manual anyway. Nevertheless knowledge of the significance of the operations he conducts and superintends is so important to a competent inspector that it might be well even in a Manual to take advantage of whatever opportunities are offered to introduce such material when it can be done briefly with telling effect.

Sampling (aggregates), p. 25

(p. 25): It is recommended, when samples are taken from conveyor belts that the conveyor be stopped. The inspector will often find that this procedure is impracticable without considerable inconvenience to the plant and resulting friction with the operator. I have nothing better to offer but suggest the desirability of research to devise an adequate method of sampling from moving belts.

Mr. Protze suggests rewriting para. 6: "If the sample is taken from a conveyor belt, the conveyor should be stopped; the entire cross-section of the stream should be taken at each time of securing material. Samples for analysis should be taken from more than one location to check both the uniformity and average quality of material. If the sample is to be taken from the discharge of a bin at least several cubic feet of the material should be withdrawn. Preferably samples should be taken while discharging several tons of material, obtaining typical samples at beginning, center, and end of discharge for purpose of checking uniformity as above. The gate should be closed each time a representative sample is selected from the static discharged material. Samples from cars . . ."

—and adding to line 4, p. 26: "The latter method allows analytical blending by trial and error without requiring further lengthy testing."

—and changing line 2, para. 3, p. 27: ". . . (such as found adhering to sugar, fertilizer or bean sacks) may cause . . ."

Principal tests (aggregates), p. 27

—and rewriting para. 4: "Principally, concrete inspection will be concerned with three general types of tests for aggregates: ((1) initial laboratory acceptance tests for suitability as to gradation, cleanliness (silt and organic impurities), strength, soundness or durability, and sometimes toughness, wear resistance, foreign substances, and composition; (2) secondary laboratory tests on approved samples to determine physical properties used in mix design, such as weight dry loose, weight dry rodded, weight damp loose, bulking, specific gravity, voids, and absorption; (3) field tests for secondary acceptance or control such as gradation, cleanliness (principally organic matter) and moisture content. The significance of the various tests on aggregates is discussed in detail in Reference 4."

Sieve analysis (aggregates), p. 27

—p. 27, para. 6, line 8, add: ". . . mesh in inches of the largest sieve required. For expedition in field checking this quantity may be reduced if the proper care is exercised. Usually . . ."

—p. 28, para. 2, line 6, add: "nor should slugs be used as sieving aids. Sieves should be maintained clean by the use of a good quality bristle brush." and in para. 3, add to last line: "sample accepted by the laboratory as representative of the material to be provided."

Of Table 2 he comments: The mechanics of the typical computations could be made clearer by replacing the footnote with two more columns in the table itself which would show the intermediate steps of "0.40 x % sand" and "0.60 x % C.A." summing up to produce the combined gradation illustrated. The arithmetic should also then be applied to the F.M. as a check, and so noted.

—and would revise p. 29, para. 2, line 4: ". . . not be given to a single test derived from stockpiled material or from tests on other aggregates made at the source of supply. These sieve analyses should be considered with such other analyses as are available in order to obtain a picture of the general characteristics of the material upon which to base allowable tolerances. Once a given aggregate (size, gradation, etc.) is selected for use, it is essential that the material available for the concrete at the mixer be uniform and unvarying within the established tolerances;" and in para. 3, line 6, add; "One test should be made for at least every 100 cu. yds. of concrete shipped;" and after para. 3: To make the Manual more valuable as a general reference and for purpose of educating the average inspector, Sections 208-S and 215-S of the Joint Committee Report should be presented here giving suggested gradations for fine and coarse aggregate.

Tests for organic impurities in sand, p. 31

After para. 3, p. 31: For reasons above, quote Joint Committee Report Sections 210-S and 216-S on Deleterious Substances.

On p. 31 several suggested amplifications: add para. 4, line 4: ". . .; it will seldom be found in properly washed sand, but may still be present if dirty water was used in washing or if insufficient wash water was used as is often the case in winter, and in dry seasons." Add para. 5, line 9: ". . . at about 4 hours; this anticipation of ultimate color should only be used with known sands on which calibration tests have been made. The color . . ." Add, after para. 5: "Although gravel has a much smaller surface area than sand, and is inherently easier to wash clean, this material is also found to contain excessive organic content in any localities, particularly with the smaller gravels. Since concrete normally contains more gravel than sand, the effect of a given organic content in gravel, as indicated by color, may be greater than with sand. Experience indicates that the use of coarse aggregate producing a colorimetric solution darker than a light yellow is dangerous to concrete strength." Rewrite para. 6, line 2: "The colorimetric test should be made each time a sieve analysis is conducted, and at least once daily. Additional determinations should be made if the amount of organic matter is near the permissible limit, or if the supply is changed, or at any other time when it is suspected that the organic content may be dangerous,—the latter to anticipate and, if possible, prevent subsequent slow hardening of the concrete." (Note that the writers have found variable organic content in aggregates one of the important causes of erraticity in concrete strengths on the job, and believe much more attention should be paid to this factor than is normally done. Such troubles

are particularly prevalent today where the "rush for defense" is too often used as an excuse for improper processing.)

Tests for moisture and absorption (aggregates), p. 32

P. 32, rewrite para 1, item 2: "If measurement of the aggregates is by weight to determine the necessary adjustments in weight to secure uniform quantities of equivalent dry aggregates in the batches."

P. 33: add to para. 3, line 3: "The tests should preferably be made in the laboratory with a precision . . ." Rewrite para. 4, line 4: "For given types of aggregate, the value is often assumed on small jobs (Reference 3) but should be accurately determined where careful control is desired." P. 34, add to para. 1, line 1: ". . . present; if the sand gradation is excessively coarse the test is of negative benefit. At the point . . ." Add to para. 4: "The engineer should in time become familiar enough with aggregates from a given source to make estimates of moisture contents (between occasional tests) accurate to 0.5 per cent."

Tests for specific gravity (aggregates), p. 34

Add para. 5: ". . . perhaps once each week to determine trends, due to changes in the pit, etc." P. 35, add para. 1:

"The bulk saturated surface dry Specific Gravity $= \frac{S}{S - I}$;

The bulk oven dry Specific Gravity $= \frac{D}{S - I}$;

where S = saturated surface dry weight of sample,
I = immersed weight of sample,
D = oven dry weight of sample."

Add p. 35, last line: "briefly in References 4 and 9." P. 36, revise para. 1: "For use with saturated surface dry aggregates, the specific gravity should be based on the surface dry condition; for materials which are to be batched in the oven dry state, the specific gravity should be based on the oven dry condition. For aggregates containing other than these two quantities of moisture, the aggregate weight should be expressed in terms of one of these given conditions (plus excess water) and the corresponding specific gravity employed. Engineers in different parts of the country usually prefer to express basic aggregate weights in one or the other of these two manners."

Tests for unit weight (aggregates), p. 36

P. 37, add para. 3, line 4: ". . . surface dry or oven dry depending on which method of mix computation is employed." Revise para. 4, line 1: "The unit weight of dry loose or damp loose aggregate is determined . . ." Add para. 6, line 5: "content is made, or the information

may be obtained from a previously constructed plot deriving from trial t' s on the given material at varying moistures."

orage and handling (aggregates), p. 39

P. 39, add para. 4, line 2: "Many specifications define this period as 24 hours minimum, but longer periods (approximately 1 week) are desirable."

Devices for batching

Change para. 5, line 2: The 2 per cent tolerance should be deleted at present date as being too inaccurate.

P. 42, add para. 1, line 3: ". . . . in accuracy, damage, or sticking materials." Add para. 3, line 2: ". . . in Fig. 19. The weighing bin should be built so that the inspector has easy view of and access to the materials for inspection and sampling. The practice of overrunning bins of one material into another should not be tolerated. Discharge gates into the mixer should be of such construction that materials can be instantly held back if desired." P. 43, para. 3, last line: "volume, as the hopper may bulge when filled, and because the effective bulking depends upon the manner or force of charging as affected by the quantity of aggregate being measured." Add para. 5, last line: ". . . of known volume. Charging of containers should be uniform; volume batching by shovel alone should never be allowed."

Batching (aggregates), p. 43

P. 43, add para. 6, last line: ". . . and surface dry. As explained above under 'Specific Gravity' one basic method of batch computation should be used throughout to maintain rationality."

P. 44, para. 1, line 3: "In any event, the weight or volume of aggregate employed should carry into the mix an amount (plus water) equivalent to the basic weight specified by the specific proportions given."

Water, p. 44

Para. 3, line 4: "Water of doubtful quality should be submitted for laboratory analysis or test: specimens of concrete or mortar (A.S.T.M. C87) containing the water should be tested for comparative strength (or durability if time is available) with respect to companion specimens made using distilled water. Potable water is usually satisfactory for use in concrete."

Mr. Crum: More information concerning the factors that influence the accuracy of water measurement (p. 45) and design of equipment to assure accuracy would be desirable. On many operations the amount of water for the batch need not be left to the discretion of the inspector and mixer operator. With frequent aggregate moisture determinations under

well controlled conditions [illegible]y the water to be added to the batch to maintain unifo[illegible]istency as well as uniform water-cement ratio c[illegible]d [illegible]efinite as the amounts of cement and aggregates. In such a case accurate water measurement is essential. One important feature of design is to make the measurement independent of variations in the pressure on the water supply line which causes difficulty on many installations. One way to obviate this trouble is by a simple arrangement of two tanks instead of one.

Mr. Protze: p. 46, add to line 2: "to incorporate this water. Promiscuous additions of water to the job-or-transit-mixer by the job personnel for sole supposed purpose of increasing the placeability of already adequately workable concrete must not be tolerated, particularly if the employed water-cement ratio is at or near the maximum allowable figure. For this reason, when concrete is proportioned by a control engineer at a central mixing or batching plant, the agitator or mixer trucks should not be allowed to carry additional water in their reserve tanks. Note that the addition of "just a pail" of water to a 4 cu. yd. load of average concrete will increase the W/C ratio approximately 0.15 gal./sack or decrease the 28 day compressive strength as much as 150 lb. per sq. in." Add to para. 2, last line: ". . . to the batch, to assure its more uniform distribution and prevention of flash setting, reduced durability, etc., which might be caused by excessive concentrations at certain points within a batch of concrete."

PROPORTIONING—Manual p. 47-52

Mr. Crum: This same discussion (referring to "Water" p. 45) applies to the topics of "Proportioning" on page 47 and "Control of Consistency" on page 65. When it is practicable to keep close track of the moisture in the aggregates as they go to the batcher there is no reason why the water measuring device cannot be set to deliver the amount of water required for each batch or group of batches.

With volumetric measurement there is probably no efficacy in such close control of water measurement but with the more accurate measurement by weight now generally in vogue greater expenditure for testing and inspection is justifiable on large work.

Close control of the batching of all four ingredients with a minimum of field adjustments not only provides uniform consistency, but also makes possible close predetermination of quantities of materials per unit of volume. If contract assurance can be given that the more expensive materials, particularly cement, will not vary more than a small percentage from the estimates considerable economy in bid prices will ensue under competitive conditions. Such minor adjustments as are necessary may

be made in the less expensive ingredients. By means of close batching of all materials paving contracts guaranteeing the use of not more than one per cent of cement in excess of the estimated amounts are being satisfactorily administered on a large scale.

Although such considerations are more in the province of the specification writer than of the inspector, inspection is better for understanding of the reasons back of the requirements.

Mr. Thomson: This chapter discusses the application of three common methods of specifying quality of concrete, namely by fixed proportions, by cement content, or by water-cement ratio. In view of the growing practice of specifying the desired quality of concrete by strength, it would appear desirable to include some discussion of methods of designing mixes to meet such a requirement.

Mr Protze: In reviewing the Manual for the purpose of discussion relative to "Concrete Control" the writers questioned whether or not the chapter on "Proportioning" should also be dealt with, but since this aspect of concrete technology forms a background for control, the following comments are presented:

P. 47, add line 1: "The principles of proportioning in the sense of mix design are discussed on page 16; the mechanics of correct proportioning is a laboratory job and/or involves analytical computation based upon laboratory background and acquaintance with the specific materials. The following discussion . . ." Rewrite para. 4: "Specifications usually state the *governing proportions* or *limiting quantities* of materials but may require instead or in addition that the concrete develop a *fixed strength*. In any case a limiting range of consistency is stated or implied."

Rewrite para. 6, line 3: ". . . and the maximum water-cement ratio are being used, the cement and water contents are increased in the given allowable proportion to provide desired slump; the yield is then correspondingly corrected by removal of equivalent quantities of sand and gravel."

Method of fixed proportions, p. 48

P. 48, rewrite para. 1, 2: "If the specifications require that the mix shall consist of fixed proportions (by weight or volume) of cement, fine aggregate and coarse aggregate (regardless of contained moisture) the water content must be adjusted from batch to batch as necessary to secure concrete of the desired consistency. This depends on the mixerman's judgment to secure uniform consistency, and then the quality (strength, etc.) of the concrete varies with variations in water content. If the aggregates were fairly uniform, which they generally are not, such variations would not be of great consequence.

"The proportions may be expressed in terms of weight, overall (loose or compacted) volume, or solid (particle) volume. The units should be clearly stated; there is considerable difference between a given proportion by weight and the same proportion by volume. Further it should be clear whether the proportions are based on dry (oven dry or saturated surface dry) or damp aggregates, and—for measurements by volume—on loose or compact aggregates. Often used bases for proportions are in terms of weight, of overall volume of dry loose or dry compacted aggregates, or of solid volume. Generally the latter is least employed and proportions designate or imply basic dry aggregates from which corrections can be made for changes in weight or volume due to contained moisture. On the other hand, as described above, fixed proportions of aggregate in terms of damp volumes results in variations due to moisture changes affecting both bulking and water carried into the mix."

Water-cement ratio method, p. 49

P. 50, add new paragraph after para. 3: "The theory of water-cement ratio (as affecting strength durability, etc.) of course applies to all cases of proportioning: i. e. the more water used per unit of cement, all other things being equal, the weaker the mix. Where methods No. 2 and No. 3 are combined, only seldom does the use of the maximum water-cement ratio and the minimum cement content produce the desired consistency. If the slump is too great, the cement content must then be maintained, the water reduced, and the yield correspondingly corrected by additions of equivalent volumes of aggregate. This condition results in an uneconomical mixture. If the slump is deficient, both the cement content and the water content must be increased, as previously explained."

Rewrite footnote p. 50: "*For aggregate, as described previously, the specific gravity, and weight per unit volume should be rational: based either on the particles in a saturated surface-dry or oven-dry condition depending upon practice common to a particular locality."

Yield; estimating quantities, p. 51

Mr. Thomson: In the example given (p. 52) for computing the cement content of a fixed proportion mix, the use of the same weight per cubic foot for sand and coarse aggregate as for cement, namely 94 lb., is confusing, as usually sand and coarse aggregate will not have the same unit weights, and neither will be the same as for cement. Also in this example, the objective of designing concrete for a given weight is unusual, as commonly the unit weight of concrete would be that resulting from the specified quantities of materials used.

Mr. Protze and associates suggest a considerable addition after the end of the present consideration of proportioning (p. 52): "The above method of finding yield is used mainly in the field as an approximate

check on the actual delivered yield of a selected mixed batch of concrete. As indicated, this method requires weighing a unit volume of typical concrete and calculating the total yield. Another method slightly less subject to error involves the actual measurement of an entire batch of concrete in an accurately calibrated box, making the necessary corrections for temperature, etc.

"*Mix computation:* For computing the quantities of materials required to produce a cubic yard of concrete, the method of yield computation is also employed. Herein the sum of the absolute volumes of the constituent materials is essentially equal to the total volume of wet concrete produced.

"The method requires previous laboratory analysis of the constituent materials for physical properties and constants. Secondly, the method requires a laboratory (or field) trial mix using these materials to indicate:

"1. necessary water-cement ratio to satisfy specifications, slump and/or strength;

"2. necessary cement-factor (or proportions) to satisfy specifications and/or slump (in conjunction with W/C ratio);

"3. necessary fine to coarse aggregate ratio to produce optimum workability and/or other qualities.

"In the absence of actual trial mixtures, previous experience with the given materials may serve as a guide in selecting sensible and workable values for the factors noted, but it must be remembered that analytical calculation in itself will not assure a rational mix: an actual trial batch must sometime be made to assure correct selection of all factors in the optimum mix.

Example: Find the quantities required to produce one cubic yard of concrete to satisfy the following requirements:

Actual W/C ratio 6.0 gal. per sack;
actual cement factor 6.5 sacks per cu. yd.;
optimum ratio of fine to coarse aggregate by dry loose volumes: $\frac{1}{1.75}$ *

*Optimum fine to coarse aggregate ratios will normally be obtained in the laboratory by trial and will usually be found to be near the following. Similar tabulations may be expressed in terms of weight or compacted volumes if desired.

Concrete Proportions in Dry Loose Uncombined Volumes	Fine to Coarse Aggregate Ratios by Dry Volumes			
	Pea Stone	GRAVEL		Crushed Stone
		¾"	1½"	
1 — 3	Special	1 — 1.8	1 — 1.85	1 — 1.6
1 — 4.5	1 — 1	1 — 1.75	1 — 1.8	1 — 1.55
1 — 6	1 — 1	1 — 1.7	1 — 1.75	1 — 1.5
1 — 7.5	Seldom	1 — 1.6	1 — 1.65	1 — 1.45
1 — 9	Used	1 — 1.5	1 — 1.55	1 — 1.4

"The following aggregate properties have been previously determined:

	Fine Aggregate	Coarse Aggregate
Weight saturated surface dry loose, lb./cu. ft.	105	95
Specific Gravity (saturated surface dry)	2.65	2.66
Absorption, per cent	0.7	1.0

Solution

Yield of one sack of cement, $\frac{94}{3.15 \times 62.4}$ = 0.478 cu. ft.

Yield of 6.0 gal. water $\frac{6}{7.5}$ = 0.800 " "

Total of above 1.278 " "

Necessary yield per sack of cement $\frac{27}{6.5}$ = 4.154 " "

Yield of cement and water above = 1.278 " "

Necessary yield of aggregates per sack ct. = 2.876 " "

Let X = no. of cu. ft. of fine aggregate (dry loose volumes) per sack of cement in final mixture. Then 1.75X is number of cu. ft. of coarse aggregate.

Yield of X cu. ft. of sand $\frac{X \cdot 105}{2.65 \cdot 62.4}$ = 0.635X cu. ft.

Yield of 1.75X cu. ft. of Gravel $\frac{1.75X \cdot 95}{2.66 \cdot 62.4}$ = 1.000X " "

Total yield of sand and gravel per sack = 1.635X " "

Thus $1.635X = 2.876$

$X = 1.76$

$1.75X = 3.08$

Proportions = 1 − 1.76 − 3.08 by saturated surface dry loose volumes.

Quantities per cubic yard:

Cement	6.5 x 94 x 1	= 611 lb.
Sand	6.5 x 105 x 1.76	= 1200 lb.
Gravel	6.5 x 95 x 3.08	= 1900 lb.
Water	6.5 x 6.0	= 39.0 gal.

"The above quantities derive from the use of materials properties for aggregates in the saturated-surface dry state. The final given weights are for aggregates in the same state. Should the aggregates dry out to the oven dry condition the final weights would have to be reduced a corresponding amount and the total water increased to allow for this consequent absorption as follows:

Absorption by sand	0.007 x 1200 lb. =	8.4 lb.
Absorption by gravel	0.010 x 1900 lb. =	19.0 lb.
Total absorbed water		27.4 lb. = 3.3 gal.
Total water to batch	39.0 + 3.3	= 42.3 gal.

"The mix could have been directly and similarly computed for oven dry aggregates, being given properties for the materials in that same state. Mix computation in terms of dry rodded volumes, would be

similar to the above except that materials unit volumes and proportions would be expressed in those same terms. Corrections for contained moisture over the saturated-surface dry state for any case will be shown later under "Concrete Control."

"Mix computation for the cases of fixed proportions by volume are less involved than the example illustrated: no unknown "X" need be assumed and the yield for each constituent may be computed directly, the total of all constituents obtained, and the cement factor computed, after which the quantities per cubic yard may be computed as above. If the quantities are to be *batched* by volume instead of weight (this is true for any method of *proportioning*) the corresponding volumes can be calculated from the total weights, and specific weights per unit volume.

"If the proportions or optimum fine to coarse aggregate ratios are expressed in terms of weight all calculations for mix quantities become obviously simpler than in the most difficult case shown above."

INSPECTION BEFORE CONCRETING—Manual p. 53-61

H. G. Farmer:* The statement (p. 56, last paragraph, first sentence,) "Form ties and spacers should be of such type that no metal will be left near an exposed surface, and should be heavy enough not to bend if workmen climb on them," does not necessarily apply in all cases as there are some types of work which permit the use of lighter form ties and spacers. Possibly the latter part of this sentence could be modified so as not to place all form ties and spacers in the same category.

Mr. Waugh: Illustrations of lack of specific details are found on pages 59, 60 and 61. On page 59 allusion is made to "minimum clear distance between parallel bars to develop bond" (last paragraph) and the reader is directed to Reference 5 for recommended values; on page 60, to support for horizontal bars, and Reference 8 is cited for specific recommendations; and, on page 61, to size of permissible opening with direction to Reference 53 for recommendations. Other instances of this type of treatment may be found throughout the Manual.

INSPECTION OF CONCRETING—Manual p. 62-79

Mr. Protze and associates would change the chapter title to "Inspection and Control of Concreting" and beginning with para. 1 make the text: "The essential requirements for satisfactory concreting are as follows:

1. The concrete should be uniform and satisfy the specification and job requirements as to workability, water-cement ratio, and quality.

*Universal-Atlas Cement Co., New York.

2. The equivalent basic quantities of materials for a given mix should be uniform from batch to batch. It follows that the materials themselves should be uniform and of satisfactory quality.

3. The mixing . . ." Add to beginning of para. 2: "The principles of concrete control in the sense of selection and proportioning of materials, and the mechanics of inspection and testing of materials have been previously discussed. Recommended practice with regard to batching, mixing . . ."

Batching of materials

Rewrite para. 5: "The essential requirement for proper batching is that the required proportions of desirable materials be obtained uniformly from batch to batch, regardless of fluctuations in the supply. In other words if two sizes of coarse aggregate are being combined to produce a composite gradation equivalent to a well graded 1" gravel, the resulting gradation must be maintained reasonably uniform by varying the relative proportions of the two materials to compensate for fluctuations in the individual gradations. For simplicity, of course, these fluctuations should be kept as low as possible. Batching is . . ."

Mixing, p. 63

P. 63, delete para. 3, since these data will be included later.

Add to para. 6, line 6: "The water line should be so located that the flow of water will not retard the charging of the mixer. In winter when hot water is used, water over 140 F should not strike the cement directly."

Mr. Julian points out, in reference to the one minute mixing time (p. 63) that "many specifications (example U. S. Navy Department Specification No. 13 Yc, Oct. 1935) require at least 1½ minutes of mixing time for small mixers and proportionately longer for larger mixers; these times are believed to be none too long.

Mr. Protze would (p. 65) add to line 3: "because there is some grinding action during mixing particularly with soft aggregates."

Control of consistency, p. 65

Mr. Crum's comments on "proportioning" (Manual p. 45) are made applicable here also.

Mr. Protze would change the last sentence p. 66, para. 1: "If it does not, the mix should be adjusted as described below." He would add to para. 2, line 6: ". . . to considerable variation particularly if improperly and non-uniformly made." He would delete from p. 67, para. 1, the last part of sentence after semi-colon, since such a procedure would decrease the yield and is then not permissible where concrete is being sold by the cu. yd. Revise para. 2, line 6: "wind, unanticipated high absorption of mixing water by aggregates, high temperature of one or more of

the ingredients, the unknown presence of an accelerator such as calcium chloride which is sometimes secretly used in bins, trucks or belts to prevent freezing, or in rare cases to the characteristics of the cement. Change in slump due to the use of high early strength cement in summertime, or in the winter with excessively hot water, should be anticipated. Concrete that has become so stiff . . ." And insert after para. 2: "*Control of Mixtures.* In addition to the general responsibilities already described for the concrete inspector on the average project, the essential duties of a "control engineer" include all such additional supervision of proportioning, measuring, and mixing of concrete to assure the production of an end product which fully satisfies the established requirements for the concrete and its constituents. Rigid control does not *assure* a perfect end product, but goes much further in this direction than does the so-called mix inspection which consists mainly of checking the cement content, keeping records, and issuing certificates. Furthermore, rigid control often pays for itself by the savings realized due to reduced allowable cement contents, more uniform and higher strengths, greater ease and economy of placing.

"Control is applicable to all methods of concrete proportioning and batching. In general the control engineer will have available (from the laboratory trial mix or from an analytical design) a basic mix to be employed on the work. The allowable tolerances in slump, water-cement ratio, cement factor, etc., will be known and the basic (saturated surface dry, oven dry, or equivalent) quantities per cubic yard will be given. The essential properties of the materials will be known, and the requirements for gradation, cleanness, etc. of the selected materials established.

"It is the duty of the control engineer previous to the start of the work and at intervals during each day, to make tests for gradation and organic content of the aggregates to check their suitability for use, and to aid in their proper proportioning to maintain uniform quantities of the basic materials in the mix. At these same periods, tests for moisture content should also be made so that the basic quantities (weights or volumes) may be corrected for these values. A good engineer will soon become proficient in estimating moisture content and adequacy of gradation between times of actual tests so that minor corrections for maintenance of slump and workability may be quickly applied when necessary. If a change in consistency is desired the mix should be redesigned or a "rule-of-thumb" adjustment applied, and the mix correspondingly corrected for yield. The following example will illustrate the points mentioned:

Given the following data on aggregates

	Sand	1" Gravel
Weight saturated surface dry loose, lb./cu. ft.	105	95
Specific Gravity (Saturated surface dry)	2.65	2.66

Absorption, percent	0.7	1.0
Required gradation	ASTM	ASTM
Required fineness modulus	2.80 ± 0.20	7.00 ± 0.20

Given the following data on mixtures

Class Concrete	Class A
Proportions, saturated surface dry loose volumes	1 – 1.76 – 3.08
Minimum allowable cement	6.5 sx/c.y.
Maximum water-cement ratio	6.0 gal./sack
Desired slump	4″

Saturated surface dry quantities per cu. yd.

Cement,	lb.	611
Sand,	lb.	1200
Gravel,	lb.	1900
Water,	gal.	39.0

Problem: to maintain rigid control of mix under normal job conditions:

Start of day:

Grading of sand satisfactory; F. M. satisfactory

Grading of gravel satisfactory; F. M. satisfactory

Organic content of sand and gravel satisfactory

The actual data for the above should be shown as a record.

Moisture contents of aggregates (by test):

	Sand	Gravel
Wet weight of sample, gms.	1000	1000
Oven dry weight	941	980
Equivalent sat. surf. dry wt.	948	990
Free water, grams	52	10
Free water, per cent*	5½	1.0

Computation of batch quantities:

[Given Saturated Surf. Dry Weights]	×	[Free Moisture Content]	=	[Free Water Carried into Mix]	,	[Saturated Surf. Dry Weights]	+	[Moisture Correction]	=	[Quantities to Batch]
Cement 611						611			=	611 lb.
Sand 1200	×	5½%	=	66 lb.		1200	+	66	=	1265 lb.
Gravel 1900	×	1%	=	19 lb.		1900	+	19	=	1920 lb.
Water 39.0		8½	)	85 lb.		39.0	—	10.2	=	28.8 gal.
				10.2 gal.						

*Moisture contents of fine and coarse aggregate should be expressed to the nearest ½ per cent since such accuracy indicates significance of tests, and moistures can be estimated to this degree.

"Assume the above batch quantities produce a workable mix of the desired slump of 4 in. After a time let us say the moisture in the sand changes to 4.0 per cent as evidenced both by the estimated drier "feel" and appearance of the sand and the drying up of the mix. An estimate of gravel moisture is still 1.0 per cent. The aggregate gradings remain essentially satisfactory as before. The mix should be recomputed on the above basis; this would give us batch quantities:

Cement	611 lb.
Sand	1250 lb.
Gravel	1920 lb.
Water	31.0 gal.

"This computation might have been more quickly made as follows: change in moisture equals 1½ per cent of 1200 lb. = 18 lb. = 2.2 gal.

1265 lb. − 18 lb. = 1250 lb.
28.8 gal. + 2.2 gal. = 31.0 gal.

In such a *correction for change in moisture* a change in water content of a mix by 10 gal. of water requires a corresponding change in *weight* of batched aggregates of 83 lb.

"Assume now that the moisture contents of the materials remain constant but it is desired to reduce the slump to 2 in. Apply the following rule:

> Each 1 in. change in slump (all other things being equal) is approximately equivalent to a change of water-cement ratio of ¼ to ⅓ gal./sack in the same direction.*

A 2 in. change in slump would require approximately ½ to ⅔ gal./sack less water, or a new water-cement ratio of approximately 5.5 gal./sack, the cement factor remaining the same, as specified. A new basic dry mix could be recomputed as before using these cement factors and water-cement ratios; until time permitted such refinement, the change in the basic mix could be instituted by rule of thumb as follows:

> To give an absolute volume of one cubic foot, is required 7.5 gal. of water, or 165 lbs. of aggregate at specific gravity approximately 2.65 . (2.65 x 62.4 =165). Stated more simply: 10 gallons of water has the same *volume* as 220 lbs. of average aggregate. This is applicable in a *correction for yield.*

New water content of dry mix = 39.0 gal. − (½ gal./sack) (6.5 sacks/yd.) =39.0 − 3.3 = 35.7 gal./cu. yd.

Add aggregates to compensate for yield:

$\frac{3.3}{10}$ x 220 = 73 lb. New basic dry mix approximately:

Cement	611 lb.	= 611 lb.
Sand	1200 + 33	= 1235 lb.
Gravel	1900 + 40	= 1940 lb.
Water	39.0 − 3.3	= 35.7 gal.

Adjustments for moisture should now be applied as before.

"A similar approach would normally be used for a desired increase in slump. In this case, however, the W/C ratio could not be exceeded, and the cement factor would also have to be increased above the minimum to allow the use of more water. In such a case the complete recomputation of mix is desirable for simplicity. Sometimes it is permissible to reduce the cement factor and maintain the W/C ratio to reduce the slump.

"Let us assume we now have returned to the original mix and the aggregates are still graded within the allowable tolerances but have so changed in gradation or particle shape to make the mix somewhat harsh.

*See also Reference 9.

The sand and gravel quantities may be adjusted to correct this condition providing the other requirements of W/C ratio, etc., are not negated.*

Condition of Aggregate Gradation		Change in Quantity per C.Y.	
Sand	Coarse Agg.	Sand	Coarse Agg.
Normal	Normal	As designed	As designed
Coarse	Normal	Add 50 lb.	Deduct 50 lb.
Fine	Normal	Deduct 50 lb.	Add 50 lb.
Normal	Coarse but well graded	No change	No change
Normal	Coarse (Harsh)	Add 50 lb.	Deduct 50 lb.
Normal	Fine	Add 50 lb.	Deduct 50 lb.

Should either or both aggregates have gradations, fineness moduli, or organic contents outside the allowable limits of the specifications, the aggregates should be rejected, the bins cleared, and desirable material made available. At times, for job expedition, the engineer may permit the use of such undesirable material if he feels that the addition of extra cement will more than compensate for the use of the inferior stock. The use of such cement penalties (approximately 10 per cent of the original cement content of the mix) shall not be used in lieu of correcting the source of error.

"Identity and color of cement should be known to and frequently checked by the engineer; a change in color might be undesirable on the job and furthermore promiscuous mixing of brands should not be tolerated where careful control is desired since at times this procedure might cause erratic setting. The engineer must recognize that different cements require varying water contents to produce the same consistency or workability or strength.

"Aggregates from various sources having different fundamental properties should not be employed interchangeably as an error in yield will probably result together with erraticity in workability.

"The operation, condition and accuracy of the batching devices should be checked from time to time. In other words the concrete control engineer must continuously observe all the individual plant operations dealing with concrete measuring, mixing, etc. for all batches, and must at all times have his fingers on the pulse of operations and be able to readily contact the job. If his sound advice is not followed, the control is weakened and then richer mixes will be required to meet the minimum strength requirements with safety."

Compacting, p. 73

Mr. Thomson: The caution (p. 74) against using the vibrator to "cause concrete to flow" might well be emphasized, as this is a common

*The following changes in aggregate proportions indicate how to keep workability constant with changes in material gradation. These corrections may be applied in combination and—if the case is very severe—up to twice the corrections shown may be applied:

misapplication of the vibration method, which may lead to dire results. It might also be suggested that the internal vibrator should be kept at least 3 ft. away from the edge of the flow.

Finishing; repairing surface defects—unformed surfaces, p. 77

Mr. Crum: The caution (p. 77) that "sprinkling dry cement on the surface should be avoided" is good advice for the most part, but the practice has some warrant in cases where the finishing is handled by the general principles prescribed in the Johnson method of pavement construction and the inspector might well be made acquainted with the conditions under which this practice should not have bad results. An important feature of this method is the absorption of excess water by the surface cement under long continued manipulation which incorporates the surface cement with the concrete and carries it past the time of initial set.

INSPECTION AFTER CONCRETING—Manual p. 80-83

Curing

Wilfrid Schnarr:* Curing might be given more importance, thus emphasizing the advantage gained in increased strength due to the proper curing of concrete. A figure similar to the one shown in "Design and Control of Concrete Mixtures," published by the Portland Cement Association (Fig. 3, page 33) might send home the value of curing which costs so little compared to the increase in strength. Some reference might be made to bleeding causing the water to rise to the surface, during which time no additional curing water should be added even after the hours stated. Where the surface is to be finished it is advantageous to remove surplus water so that the surface hardening will be more uniform.

P. 81 (fifth paragraph, line 4) a time limit of one hour is suggested; and in the last line on the same page it mentions that spraying within the first few hours after concreting may wash some cement out of the concrete surface. Would it not be better to eliminate actual time limits as so many varying factors affect the time when curing should be started for different methods of curing?

M. A. Swayze†: The matter of concrete curing is of prime importance, and the paragraphs under that heading in the Manual all represent what we think of now as good practice. Therefore, what I have to say must not be construed as criticism of the Committee, since they have presented only generally accepted methods.

Over the last two or three years our laboratory has been studying early volume changes in concrete. In that study the matter of curing was inevitably involved, and from what we found it seems to me that we must do a lot of more serious thinking about this subject than has

*Supervising Concrete Inspector, Hydro Electric Power Comm. of Ont., Toronto.
†Director of Research, Lone Star Cement Corp., Hudson, N. Y.

been done. I will illustrate by citing some of the present instructions. Here is one of them: "The preferred method of curing is the use of continuous sprays—The spray shall be applied to unformed surfaces one hour after placing." In other words, the concrete in such a case will have all the water it can possibly drink in, almost from the time when the spray could wash the cement out of the surface.

A little further on the topic is wet burlap. You can't put burlap on quite so soon, since the concrete has to be finished first. This means waiting for 2½ or 3 hours after placing under normal conditions.

Still further, under bituminous coatings, "The surface to be sealed should still be moist when the coating is applied." In other words, not only should no curing water be added, but you should so fix the concrete surface that it can get no further curing water whatever after the coating is applied.

When waterproof paper is used for curing, apparently its function is merely to prevent evaporation from the surface. We are admonished to take an occasional peek under the paper, and if the surface is dry, to rewet it.

Here are four different methods for curing concrete. One says start curing at the age of one hour, another implies that about three hours is right, a third calls for no water at all, while the fourth says wait until the surface is dry and then wet it. Obviously if one method is right, then the rest are wrong. I have an idea what we should do is to go light on curing water at early ages, and then give the concrete all it will take after an age of 12-24 hours.

I will have more to say about this at another time*, but it seems clear that this section of the Manual should be given some very earnest thought as to which is really the right way to cure concrete and which is the wrong way.

TESTING OF CONCRETE—Manual p. 84-95

Mr. Thomson: In the introduction to the chapter on "Testing of Concrete," there might be included an explanation of the two main objectives for making tests, namely either to check on the quality of the concrete or to check on the probable development of strength in the actual structure at a given time. It should be pointed out further that if verification of the quality of the concrete is the objective, then it is necessary that the specimens receive standardized treatment as outlined in the ASTM specifications. This is especially necessary with high-early-strength concrete, because any variation in the temperature or moisture

*See "Early Concrete Volume Changes and Their Control," M. A. Swayze, ACI JOURNAL, Apr. 1942; *Proceedings* V. 38 p. 425.

experience of the specimen may have a marked effect on the strength observed, and may result in incorrect conclusions as to the quality of the concrete.

Number of specimens, p. 85

Mr. Crum: The statement (p. 85) respecting the number of specimens required indubitably reflects the best current practice, but I cannot refrain from pointing out that the correct number of specimens really depends upon the required accuracy in the average of the tests in relation to the true value of the average of the tests of all of the specimens into which the unit of material being inspected might be divided. This relation is statistically determinate and for practical use waits merely upon the accumulation and study of data that will establish the statistical characteristics of various kinds of concrete and concrete materials (see Manual Ref. 4).

RECORDS AND REPORTS—Manual p. 96-103

Mr. Julian: A sample set of reports covering the following, it is believed, would be a valuable addition to the Manual: (1) Mechanical analysis of aggregate, (2) Strength test of fine aggregate, (3) Strength test of coarse aggregate, (4) Soundness test of fine aggregate, (5) Soundness test of coarse aggregate, (6) Test of cement (strength, set, compound composition and chemistry), (7) Volume change and soundness test of cement, (8) Daily log of concrete operations, (9) Reports of tests on concrete.

Since the Manual covers the reasons for a number of phenomena, a discussion regarding retrogression in strength such as is illustrated by Fig. 97 (pg. 314) of "Materials of Construction" 5th edition by Mills, Hayward & Rader, might not be out of place. A frank discussion of this phenomena and what steps should be taken to avoid it would be most refreshing and interesting. It appears to be a much neglected subject.

SPECIAL METHODS OF CONCRETING—Manual p. 104-117

Cold weather construction, p. 104

Mr. Thomson: The statement in the last paragraph p. 104 that "the casting of ordinary concrete structures at temperatures below about 40 F. is considered undesirable" is quite misleading. If unheated concrete is used, this is substantially true. But at present, in most of the larger cities where severe freezing occurs, there are available either heated aggregates for job-mixing, or heated ready-mixed concrete which can be furnished with a temperature of 60 F. or above; both arrangements will enable placing of concrete at much lower temperature than 40 F. to

proceed with entire safety, and with little extra expense for protection of the cast structure against freezing. In fact, it is much cheaper to heat the concrete prior to placing and thus obtain normal setting characteristics, than to place it in cold condition and attempt to heat it from the outside. Much concreting is now carried on in freezing weather by use of warm concrete.

On p. 105 appears the statement: "the temperature of the mixing water as it comes into contact with the cement should not exceed about 140 F." This might be interpreted as meaning that the water should not be heated above 140 F. Such a restriction is not in line with much of the practice and appears unnecessary. The suggestion might be added to the text that when hot water is used, the water and aggregates should enter the mixer prior to introducing the cement, which procedure will reduce the temperature of the water to a point which will not cause rapid setting, because the temperature of the aggregates, even when pre-heated, will be much below 140 F.

Ready-mixed concrete, p. 106

In the discussion p. 106, reference is very properly made (No. 30, p. 119) to the "Recommended Practices for Truck Mixers and Agitators" adopted by the National Ready Mixed Concrete Association; the recommendations contained in the other references at this point, No. 5 and 35, are similar in intent. But in view of the current extensive use of ready-mixed concrete in metropolitan areas, and the likelihood than an inspector may not have a copy of Reference No. 30 at hand, it is suggested that the text might include a summary of the recommendations as to capacities of bodies and also rate and number of revolutions. The text might also refer to the necessity of operating truck mixers and agitators in a reasonable manner as intended by the manufacturers, and avoiding any confusion with methods of operating stationary mixers. There have been many serious abuses of ready-mixed equipment as a result of inspectors demanding improper operation.

Mr. Cohen: The point Mr. Thomson made of the interference with concrete operations would not happen if you had an inspector engineer who played around with materials enough to know or see, just by visual inspection, that the quality of the concrete was lacking. It would not be necessary to stop the plant, and cooperation with the contractor could be obtained at all times. The better the inspector, the better the job and the better relations you have between the contractor and the job.

Mr. Protze would add p. 107, para. 3, line 7: "Enough loads of transit-mix concrete should be completely mixed at the batching plant so that the inspector will have an adequate conception of the consistency to apply proper control."

Floors

Mr. Farmer: The references on this subject are good, but it might be desirable to include in the Manual some of the information given in the references so as to be of greater value to the inspector in the field. In addition, practices which often produce unsatisfactory results should be more fully explained.

REFERENCES—Manual p. 118-121 AND STANDARD SPECIFICATIONS AND TEST METHODS—Manual p. 122-128

Mr. Thomson: The excellent lists of References and Standard Specifications include several items which have been revised or re-written at a date later than given. It is suggested that in case of any subsequent edition of the Manual, these lists should be checked carefully to insure that the latest issues of all recurrent items are shown.

Mr. Waugh: This list (pp. 122-128) is a very convenient reference, however, strictly speaking, it is not of very great value to the inspector. It appears to be of more value to engineers, architects and contractors, and therefore deserves the places it has in the Manual. A valuable addition to the Manual, so far as the inspector is concerned, would be to include in an appendix, the complete specifications and test methods which the inspector has to use in his daily routine. The list in the Manual includes 121 references, 15 of which cover the details of tests which the average inspector is called upon to make every day. These details are already available to the inspector in other publications, but the writer likes to visualize the Manual as the inspector's "constant companion" on the work—a single source of information to which he can go for specific information when he requires it. The writer has numbered this list, for convenience in this discussion, continuously down each page, beginning on page 122. The 15 items which would cover the inspector's needs, most of the time, are, 1, 14, 15, 16, 17, 18, 19, 20, 27, 28, 32, 94, 95, 98 and 100.

The Manual as it stands, is an excellent beginning toward the fulfillment of a distinct need, and is a valuable aid to the inspector. The committee and Mr. Kelly are to be commended for the excellent work they have done in producing the Manual.

CLOSURE FOR COMMITTEE 611

J. W. Kelly:* The assignment of the Committee, as we understand it, was primarily to prepare this Manual of Concrete Inspection. However, by implication at least, we understand that we are to promote in any way we can the general problem of getting competent inspection and more of it wherever it is not now provided adequately. That is perhaps more

*Civil Engineering Dept., University of Calif.; Author-Chairman Committee 611, Inspection of Concrete

important than the Manual itself. This Manual is but a means to an end; if it is not used it has failed in its purpose. One way to start the general use of the Manual is for the men who have inspectors to engage, train, and back up to read at least portions of this Manual and to act accordingly. We ask the entire membership to encourage the provision of inspectors and inspection, the training of these inspectors, and the raising of the status of the inspector to a real profession that more men will get into and stay in, with pride in their work. In this way I think we will accomplish more than if we devote our attention merely to a pamphlet which tries to set forth the things that an inspector should do.

We hope to improve the Manual and to make it more effective. The membership of this Committee was chosen from widely diversified interests and has given us many viewpoints which necessarily must be compromised in any statements about inspection. The Manual is not the work of the Chairman alone; some members of the Committee contributed in very large measure and in some cases drafted or re-drafted certain sections. Discussion of the Manual will undoubtedly continue to be open until it goes to the Institute, in revised form, for adoption as a standard. We welcome discussion, and all discussion will be given full consideration by the Committee. Of course we cannot adopt all of these suggestions, even if we see the viewpoint of the discussor, because in many cases the discussion or proposal of one man will conflict with that of another.

A word about Committee policies: The coverage of inspection is so broad that outside of structural design we attempt to cover almost everything in the field of concrete, that is, in the field of construction. We could not put that all in detail in a small enough book that a man would read, so we have relied on certain other documents which we feel the inspector should have. For example, no man who claims to be an inspector should be without the little booklet, "Design and Control of Concrete Mixtures." If he is concerned with structural work, he should have a copy of the Joint Committee report. There are other pamphlets equally valuable which should be in the library of any forward-looking inspector.

Another policy of the Committee is that we are not to *make* policy on matters of construction practice but that we are to *interpret* the policies set forth by other authorized bodies. There is a committee on the Design of Concrete Mixtures, a Committee on the Measuring, Mixing, and Placing of Concrete, and so on; and we cannot undertake to set up ideas differing from those and other committees which have given special attention and special study to certain phases of concrete. Some matters of disagreement with the Manual should therefore be considered by the appropriate Committee, and if they agree to the suggested changes this Committee of course will follow.

The general reaction to the Manual has been very good; we feel proud of the fact that we have had more bouquets than brickbats. The voluntary discussion was very light; except for one paragraph on hot cement, there was no real complaint on the contents of the Manual as it was published. Prior to the convention, we asked some twenty members to discuss the Manual and assigned particular sections for special attention. Some of these members offered no discussion. Some, in order not to be accused of being inattentive, wrote that they had taken particular pains to read the Manual with the idea of finding criticism, but had nothing to offer of special significance. Others, in response to this request, submitted criticisms and suggestions at the convention and in the JOURNAL.

Of course there are a great diversity of ideas as to what a Manual is and what a Manual should contain. The Committee's concensus was that this Manual should be a small book which is not so long that a man would not read it, but not so brief and direct that an uninformed man would try to follow the rules without knowing why he was doing so. A very specific example of the failure of a type of instruction which would simply tell a man what to do is the old A.S.T.M. tentative rules for concrete inspection, published in 1921, carried in the Standards for some ten years, and finally dropped from the A.S.T.M. Standards—an excellent document and really the nucleus of this Manual. It said the inspector shall do this and shall do that, but so far as we are able to learn, very few people ever used it. Necessarily then, what we have produced is a compromise, not so lengthy as some would like it and not so short as others would like it.

The discussions can be classified into three general types: those few which call for less in the Manual, those which call for change, and those (the most numerous) which call for additions. We have had a similar situation in the Committee.

Regarding the proposed shortening of the Manual, one suggestion is that it is too much of a manual of construction, that we have not confined ourselves to inspection but have also told in many cases how to construct. A suggestion also has been made that we might separate the what-to-do from the how-to-do-it and why-to-do-it. Personally I see no possibility of doing that satisfactorily, but if any one will give us a suitable sample of a page or so, the Committee will consider the matter again.

Second, in regard to changes: We received some well-founded objections, one paragraph about hot cement, which was then changed in the second printing. Another discussion, at the convention, proposes a change in our policy of dealing with the water-cement ratio. There are also many proposed changes in detail, most of which are undoubtedly acceptable.

Now as to the third type of discussion which would add something to the Manual, I am glad to say that most of the discussions are in this

group. If we were to conform to all of the written discussion we would add many pages. Some of these suggestions cover matters which are not within the province of this Manual, but they are all welcome and will be accepted to the extent to which we can incorporate them.

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PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

VOLUME 38—1942

From JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, Vol. 13, Sept. 1941, to June 1942 and Supplement, November 1942.

This is an Index of:

Original contributions to the JOURNAL OF THE AMERICAN CONCRETE INSTITUTE—papers, reports, discussions by subject, title and author.

For the convenience of the reader who is referring to JOURNAL issues rather than to Bound Volumes (in which discussion is assembled following the paper discussed) there is a reference to JOURNAL issue in which discussion appeared. This reference to discussion is by supplementary page numbers. If the last page of a paper is 28, the first page of the discussion, published later, is 28 - 1.

In general, important subjects are classified and indexed under approximately 30 main headings each one appearing in its proper alphabetical order in bold face capital letters—as for instance, ARCHITECTURAL DESIGN. "Housing," and other subjects classified under this head, are indented.

Key words to important subjects, appear, in alphabetical order in addition to the general classification—as for instance "Admixtures" and "Aggregates" each referring to MATERIALS AND TESTS under which all allied references appear, indented. Authors' names and original titles of papers appear in bold face type in proper alphabetical sequence with the subjects, with references to their contributions.

Specific data on Beams are so indexed by reference to ENGINEERING DESIGN or TESTS OF MEMBERS AND COMPLETED STRUCTURES thus avoiding an oversight by the searcher of important allied data.

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